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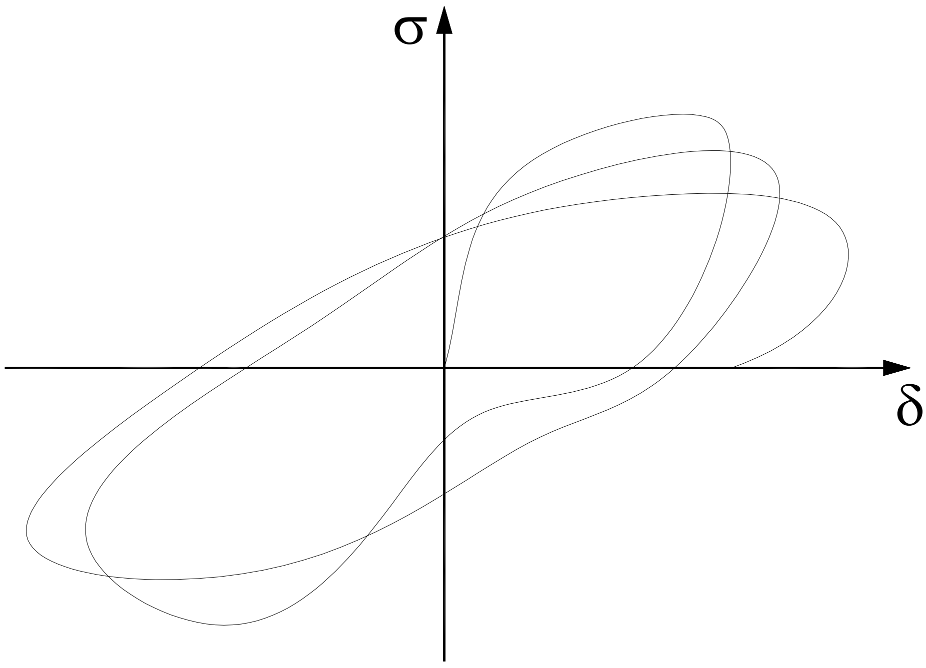

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Preface

Seismic design procedures were first incorporated in building design codes in the 1920s and 1930s when inertial loadings began to be appreciated. In the absence of reliable ground measurements during an earthquake as well as detailed knowledge of the dynamical response of structures, the seismic action was taken into account for design purposes as a statical horizontal force corresponding to about 10% of the weight of the structure.

By the 1960s ground measurements during an earthquake in the form of accelerograms were becoming more generally available. At the same time the development of strength design philosophies and of computer-based analytical procedures such as the spectral-modal analysis and the time-history analysis, facilitated the examination of the dynamical response of multi-degree-of-freedom structures (MDOF). According to these procedures, the calculations were carried out in a deterministic fashion. The response of the structure was assumed to be in the elastic range and the earthquake loading was taken into account considering the typical seismic intensity and soil nature of the site of the structure.

As the records of strong ground motion were increasing, it became apparent that the code provisions were inadequate in providing the required structural strength of the building to withstand an intense earthquake. This was recognized analysing the damage in structures that had been close to resonance. In fact the vibrating masses of structures in such a situation had often been exposed to accelerations from two to six times the maximum base acceleration that, of course, would induce forces on the structural elements much larger than expected in the design phase. However the lack of strength did not always result in failure and sometimes not even severe damage. On the other hand in specified regions of the structure (especially the ones with shear dominated behaviour) a rapid reduction in strength (brittle failure) was observed leading to local failure that often resulted in the formation of mechanisms and consequently collapse of the structure.

This type of observations called the attention of structural engineers to the property of the materials or of the structures to offer resistance in the inelastic domain of response. This property is generally known as *ductility* and includes the ability to sustain deformations in the inelastic range without significant loss of strength and a capacity to absorb energy by hysteretic behaviour.

Consider the simple case of a single degree of freedom (SDOF) oscillator. Assuming a fully elastic behaviour, *case a)* in Figure 1, it is necessary for the oscillator to develop a strength of S_E to reach a level of deformation equivalent to δ. However the system does not have to be that strong to reach the same level of deformation ($S_{EP}<S_E$) provided that can undergo inelastic deformations without significant loss of strength so that an elasto-plastic behaviour may be assumed, *case b)* in the same figure.

Consider now for both cases the energy stored at deformation level δ given by the area between the load-deflection curve and the deflection axis. For the elastic response the energy is totally converted into kinetic energy (area *abc*). For the elasto-plastic case when the mass returns to zero load position the kinetic energy is represented by the small area *efg*. A significant portion of the energy at deformation level δ, given by the area *adeg* is dissipated, i.e. converted in other forms of energy such as heat.

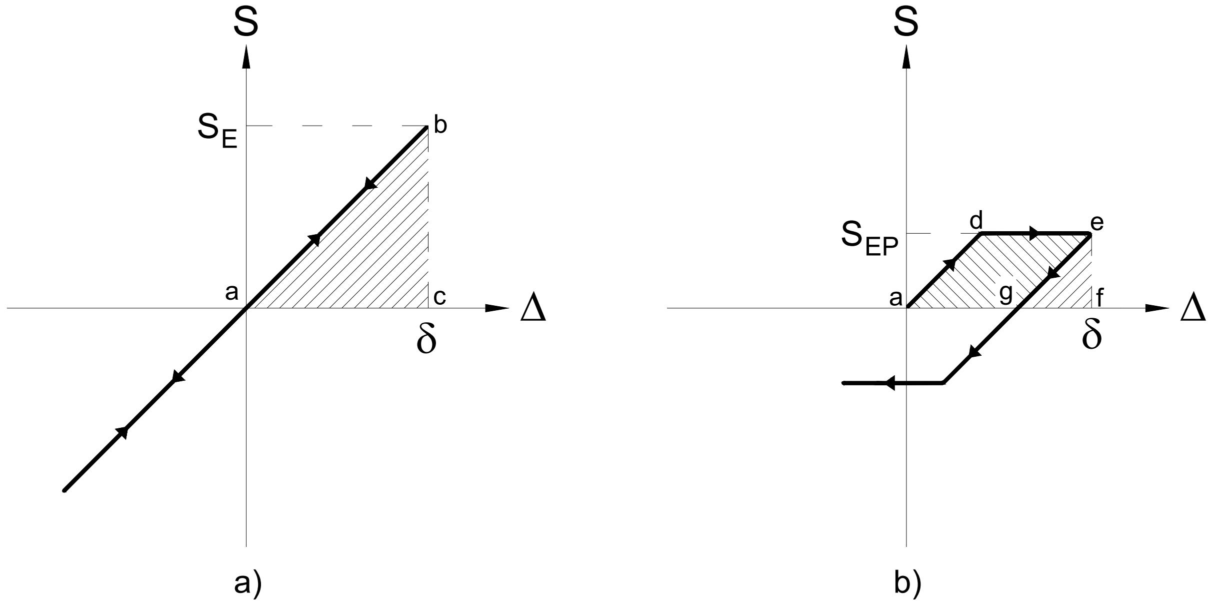

Figure 1 - Response of a SDOF system: a) Elastic response and b) Elasto-plastic response (Park and Paulay, 1975)

The awareness of these features together with the impossibility of predicting the characteristics of the ground motion to which the structure will be submitted and the economic unfeasibility of designing in the elastic range shifted the emphasis in design from the resistance of large seismic forces to the "evasion" of these forces by means of a rational distribution of ductility and strength in order to ensure a desirable mode of behaviour regardless the type and intensity of the earthquake. Nowadays, ductility is considered the most important single property sought by a structural engineer when designing in regions of significant seismicity as it gives the designer the choice to design the structure for much lower forces than the consideration of an elastic system would require.

The desirable behaviour of the structure may be achieved if regions where the energy from the earthquake may be dissipated are rationally chosen and if significant loss of strength and brittle behaviour are prevented through a proper design procedure, so that a predicted "energy dissipation mechanism" would hold throughout the seismic action. These regions are generally known as *yield zones* or, in frame structures, *plastic hinges* and their main task is to dissipate the energy by means of stable incursions in the inelastic range. All other structural elements are then protected against actions that could cause failure, by providing them with strength greater than that corresponding to development of the plastic hinges. In this way the chosen means of energy dissipation are maintained and the structure may survive an earthquake with limited damage. This design concept is the base of modern code provisions and is generally known as the *capacity design procedure*.

The area of greatest uncertainty in the response of capacity-designed structures lies in the level of inelastic deformations of the plastic hinges both regarding the extension of these deformations and their maintenance throughout the seismic action. To apply this design philosophy, it is of primary importance to define and to understand the behaviour under cyclic loading of all the elements composing the structural system in order to decide which energy dissipation mechanism is to be used: On one side the yield zones should be provided with enough ductility so that they can effectively dissipate energy from the earthquake and maintain their strength. On the other side the rest of the structural elements should be prevented of reaching lower strengths than the ones corresponding to the development of the yield zones. This means that on one side it is necessary to understand the means by which a ductile behaviour may be achieved and on the other side to understand the trends in which brittle behaviour develops and may be prevented.

Therefore the primary objective of this paper is to present a review of today's knowledge about the cyclic behaviour of reinforced concrete structural elements namely linear members and beam-column joints, i.e. frame elements. Complete and extensive information may be found in the list of references at the end of the document. It is the author's intention that the information provided serves to understand and evaluate the different provisions of the modern codes on Seismic Design. Therefore emphasis is on analysis rather than on design. Relevant experimental data will be provided whenever possible aiming better illustration and understanding of the subjects to be exposed.

Acknowledgements

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The author would like to thank his supervisor for giving valuable advice and motivation as well as important criticism to the present work.

A word of appreciation must also be addressed to the author's co-supervisor M.Sc. Ph.D. Rita Bento, Instituto Superior Técnico, Lisbon, Portugal, for her prompt availability in providing help in the form of important and useful comments and suggestions.

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Lyngby, November 2003

João Luís Domingues Costa

Contents

1. Properties of Reinforcement and Concrete

1.1. Concrete

One of the most important features influencing seismic behaviour of reinforced concrete is confinement. This feature refers to the influence that lateral reinforcement (in the form of hoops or spirals) has on concrete namely the favourable effect on ductility and strength. However, before discussing the properties of confined concrete it is important to bear in mind the main features of the behaviour of unconfined concrete.

1.1.1. Unconfined Concrete

Monotonic loading

Diagrams of stress (σ) versus strain (ε) for monotonic compression and for various concrete grades are depicted in Figure 1.1. These diagrams result from tests on cylinders and were carried out with deformation control after the development of the maximum strength (CEB, 1993).

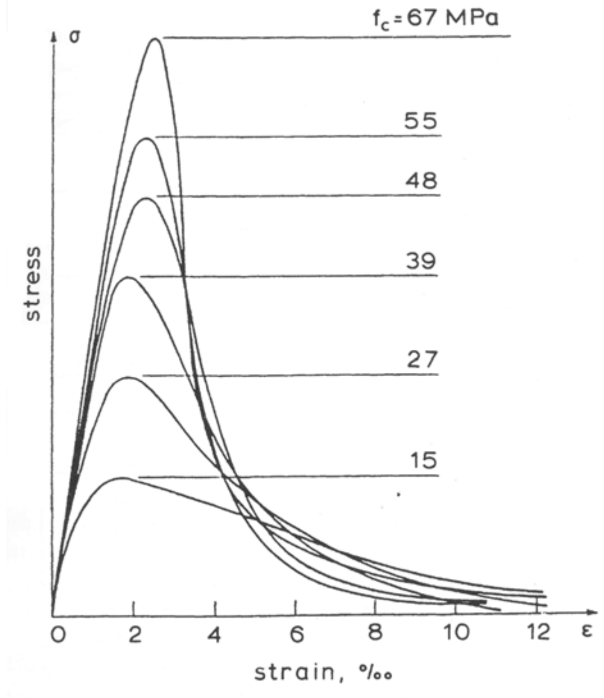


Figure 1.1 - Monotonic stress-strain relationship for compressive load (CEB, 1993)

A study of these curves leads to an important conclusion: Low grade concrete is more ductile than high-grade concrete. In fact as the concrete strength increases the descending branch gets steeper, indicating a brittle behaviour. This apparent brittleness in high-strength concrete is of great concern and it must be considered when a concrete structure is subjected to high compression strains.

It should be noted that these tests are of the statical type. During an earthquake strains may vary at a rate of 1-2% per second. It is known that the concrete compressive strength under dynamical loading, $f_{c,dyn}$, exceeds up to 20% the monotonic compressive strength for strain rates, $\dot{\varepsilon}$, of the order 1% per second in normal grade classes being this value lower for higher classes. On the other hand, large strain rates lead to a steeper slope of the descending branch of the stress--strain diagram. This implies that high strain rates, such as those of seismic loading, have positive, as well as negative, effects on the response of concrete.

Three different parts can be clearly identified in the monotonic curves shown in Figure 1.1:

1) An initial part with a linear branch indicating elastic behaviour.
2) A second part, for strains corresponding to stresses of 70% to 100% of the maximum strength, where a gradual reduction in stiffness is evident. In this range of stresses bond cracks in the interface of the mortar and aggregates, due to the difference of stiffness of the two materials, start to develop into mortar cracks mainly due to stress concentrations at the tips of the bond cracks.
3) A third part for strains larger than the one corresponding to maximum strength. This is the above-mentioned descending branch indicating the so-called strain--softening phenomenon. In this phase the internal cracks propagate in an unstable manner and tend to become a macroscopic phenomenon.

An important parameter for strength and ductility calculations is the ultimate compressive strain, ε_{cu}. For design purposes this parameter is defined by the value at which the maximum bending capacity for a cross-section is achieved. Most of the codes range this parameter from 0.35% to 4.0%.

Because of the low value of the tensile strength, compared to the compressive strength, and because the seismic action induces significant inelastic response on structural elements and pronounced tensile softening due to cyclic loading, this parameter is not usually taken into account in strength calculations for seismic design. If tensile stresses are considered the stress-strain relation of concrete in tension may be defined as a straight line up to the tensile strength. The corresponding modulus of elasticity is taken the same as in compression.

Response to cyclic loading

With the intention to study the behaviour of unconfined concrete under alternate compression, Karsan and Jirsa, 1969, carried out an experimental work that led to the results presented in Figure 1.2. This figure shows the stress-strain diagram of a concrete cylinder subjected to repeated uniaxial compression involving loading and unloading under deformation control.

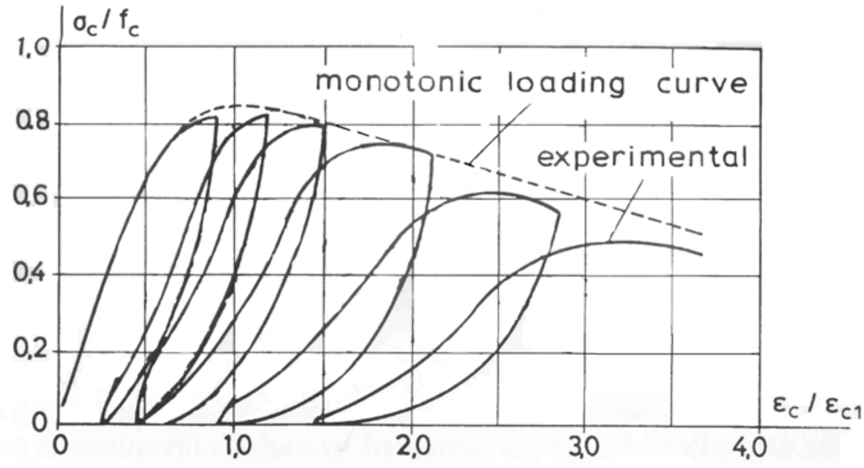


Figure 1.2 - Cyclic uniaxial compression with full unloading (Karsan and Jirsa, 1969)

Until the maximum strength is achieved, the loading branch coincides with the monotonic loading curve. For the unloading branches, two distinct parts can be pointed out: the one immediately after the maximum stress, which is extremely steep due to the highly compressed state of the concrete at early stage of the unloading phase, and the following with a minor stiffness as a result of the plastic deformations formed in the previous cycle. Regarding the reloading branches, the reduction in slope and in the maximum strength with the number of cycles may be observed. This is due to successive degradation of the internal structure of the specimen caused by the propagation of the mortar cracks, after each cycle.

A first conclusion from the results obtained is that repeated high-intensity compressive loading produces a pronounced hysteretic effect in the compressive stress-strain relationship of concrete. This is evident observing the slope and strength reduction of the successive unloading and reloading branches after each cycle. Thus, as the number of loading-unloading cycles increase, the compressive strength and stiffness of the concrete decreases, indicating the softening of the material with alternate loading.

It can also be seen that the envelope curve (the limiting curve below which all stress-strain curves lie) is almost identical to the monotonic loading curve. This conclusion is of particular importance when modelling the response of concrete to uniaxial compression. In fact, for this state of stress and for practical purposes, the most important aspect in modelling is the accurate description of the envelope curve rather than the detailed shape of the reloading and unloading branches.

Aoyama and Noguchi, 1979, concluded that alternate loading-unloading does not affect the behaviour of concrete as long as the imposed compressive stress, σ_c, does not exceed about 50% of the dynamic strength in compression, $f_{c,dyn}$. On the other hand, if $\sigma_c \geq 0.85 \cdot f_{c,dyn}$ significant reduction both in compressive strength and stiffness, as in Figure 1.2, must be expected due to the successive spreading of the mortar cracks.

For the study of the response of concrete to cyclic loading, concrete in tension has no practical significance. As soon as the tensile strength is exceeded, cracking will take place and so energy dissipation through hysteretic loops will be negligible. When a structure is subjected to an earthquake this phenomenon might well happen right in the beginning of the loading. Therefore, the effect of tensile strength is usually disregarded for seismic design purposes.

1.1.2. Confined Concrete

It is known that both ductility and strength of concrete significantly increase in a triaxial compression state of stress. In practise this stress field may be approached by providing adequate lateral reinforcement as long as it prevents the element from lateral expansion when subjected to axial compression.

The effect of confinement depends on the level of the lateral expansion, which is directly related with the compressive stress by means of the *Poisson effect*. At low levels of compressive stress, the lateral reinforcement is hardly stressed and therefore the concrete is considered unconfined (first part of the monotonic curves of Figure 1.1). The concrete becomes confined for levels of compressive stress close to the uniaxial compressive strength. At this stage (second part of the monotonic compressive stress-strain relationship) the lateral expansion resulting from the spreading of the internal cracking activates the lateral reinforcement, which then leads to a confining reaction to the concrete. In this way lateral reinforcement provides passive confinement preventing the unstable propagation of the internal cracking.

Thus, the favourable effect of confinement is due to the fact that transverse pressure from lateral reinforcement keeps the inner structure of the concrete member preserved delaying the failure due to sliding along the cracks.

Experimental work carried out by Scott, Park and Priestley, 1982

The above-mentioned authors conducted an experimental investigation on the behaviour of short reinforced concrete columns submitted to failure in compression at different strain rates, $\dot{\varepsilon}$, ranging from 3.3x10^{-6} s^{-1} (static loading) to 16.7x10^{-3} s^{-1} (seismic loading). The specimens were 450 mm square by 1200 mm high and contained either 8 or 12 longitudinal reinforcement steel bars and different arrangements of square steel hoops (Table 1.1). A specimen with the same dimensions but with no reinforcement at all was also tested for comparison.

Table 1.1 - Details of tests specimens and test results (Scott, Park and Priestley, 1982)

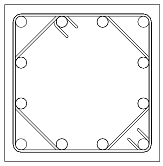
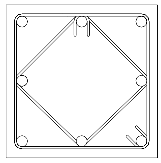
Specimen number		1	12	13	14	15	17	18	19	20
Concrete compressive strength (MPa)		25.3	24.8				24.8			
Reinforcement arrangement		-								
Longitudinal reinforcement	Diameter (mm)	-	20				24			
	Yield strength (MPa)	-	434				394			
Transverse reinforcement	Diameter (mm)	-	10		12		10		12	
	Spacing (mm)	-	98	72	88	64	98	72	88	64
	Yield stength (MPa)	-	309		296		309		296	
	Volume ratio of transverse steel, ρ_w (%)	-	1.40	1.82	2.24	3.09	1.34	1.74	2.13	2.93
Strain rate, $\dot{\varepsilon}$ ($^{o}/_{oo}$)		3.3×10^{-3}	16.7				16.7			
Maximum strength (MN)		4.38	8.50	8.65	8.80	9.40	7.90	8.50	8.40	8.80
Average concrete strain at first hoop fracture ($^{o}/_{oo}$)		-	0.03	0.04	0.045	0.055	0.04	0.025	0.035	0.04

Figure 1.3 shows the stress-strain diagrams for specimens 1, 17, 18, 19 and 20.

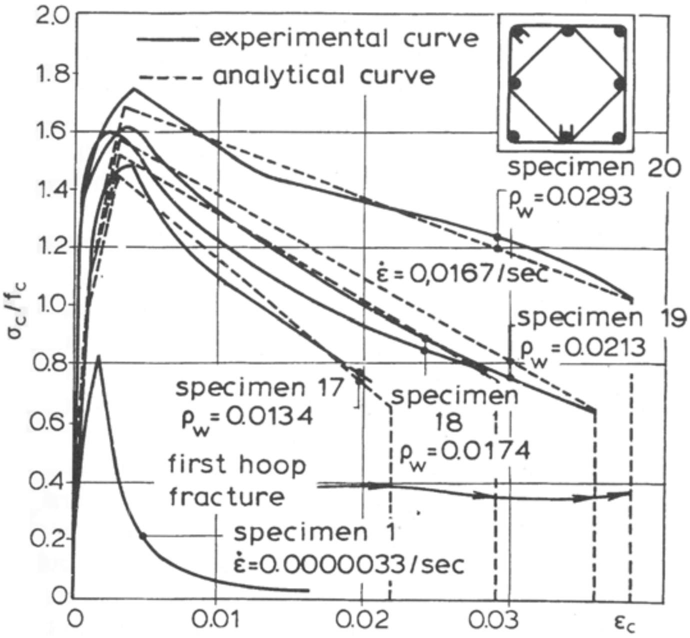


Figure 1.3 - σ_c - ε_c diagrams of unconfined concrete specimen and specimens with different hoop configurations (Scott, Park and Priestley, 1982)

It should first be pointed out, as mentioned above, that the strain rate for the confined concrete specimens corresponded to seismic loading, $\dot{\varepsilon} = 1{,}67\%$, while the plain concrete specimen was tested under static loading. The strain rate affects the response of confined concrete in the same way as explained for the case of unconfined concrete (Larger maximum strength, but steeper descending branch in the σ_c - ε_c relationships). Therefore, to accomplish a direct comparison, the ordinates of the σ_c - ε_c relationships for the confined specimens should be reduced by about 24%.

This experimental work clearly highlights the main advantages of confined concrete over unconfined concrete:

- Confined concrete has a significantly larger compressive strength. The compressive strength of the plain concrete did not exceed 86% of the cylinder strength, f_c. On the contrary, confined concrete compressive strengths reached values from 19 to 41% higher than f_c after the above-mentioned adjustment. It should be noticed that under seismic loading (high strain rate) the concrete strength may reach values up to 80% higher than the cylinder strength.
- Confined concrete has a significantly larger reserve of ductility, which for seismic design purposes is of primary importance. In these tests the strains were measured until fracture of the first hoop. The values recorded for the ultimate strain ranged from about 25 to 40$^{o}/_{oo}$, which is an order magnitude higher than the values usually obtained in unconfined concrete (3,5-4.0$^{o}/_{oo}$).

Types of Confinement

Mainly two types of lateral reinforcement are used to confine concrete: circular steel spirals (Figure 1.4 – a) and square or rectangular steel hoops (Figure 1.4 – b). Tests (Aoyama and Noguchi, 1979) show that spirals are more effective than rectangular hoops regarding the favourable effect on ductility and strength.

The reason for the considerable difference in confinement by these two types of lateral reinforcement lies in their shape (see Figure 1.4.).

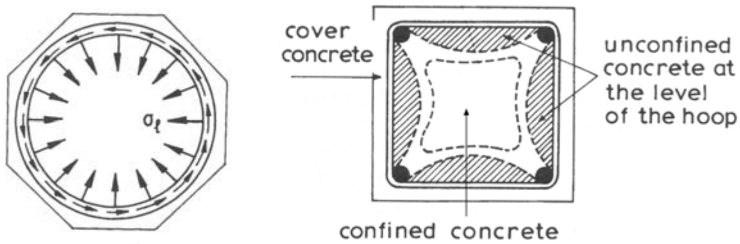


Figure 1.4 - a) Circular spirals b) Square hoop (Penelis and Kappos, 1997)

Circular spirals are in axial hoop tension and provide a continuous confining pressure. When closely spaced they provide a state of stress near triaxial compression at large transverse strains. On the other hand, square or rectangular hoops can only provide confinement in the region near the longitudinal reinforcement bars and in the centre of the cross-section, because the lateral expansion of the concrete tends to bend the sides of the hoops outwards. Thus a significant portion of the cross-section is unconfined.

Parameters affecting confinement (Experimental work carried out by Ozcebe and Saatcioglu, 1987)

With the intention to show the influence of confinement on the cyclic behaviour of concrete, G. Ozcebe and M. Staacioglu tested 4 full-scale columns under simulated seismic loading. However, in this work only the results corresponding to three specimens (U3, U4 and U6) are discussed. The experimental program is depicted in the Figures 1.5 and 1.6 and in Table 1.2.

Table 1.2 - Properties of the columns (Ozcebe and Saatcioglu, 1987)

Test Specimen	*Concrete Strength (MPa)*	*Longitudinal Steel*		*Transverse Steel*				
		$f_{y,l}$ *(MPa)*	ρ_l *(%)*	$f_{y,t}$ *(MPa)*	ρ_w *($^o/_{oo}$)*	*s mm*	*Configuration*	$\frac{A_w \cdot f_{y,t}}{s}$ (N/mm)
U3	34.8	438	3.27	470	16.9	75	Type A	1253
U4	32.0	438	3.27	470	25.4	50	Type A	1880
U6	37.3	437	3.27	425	19.5	65	Type B	1262

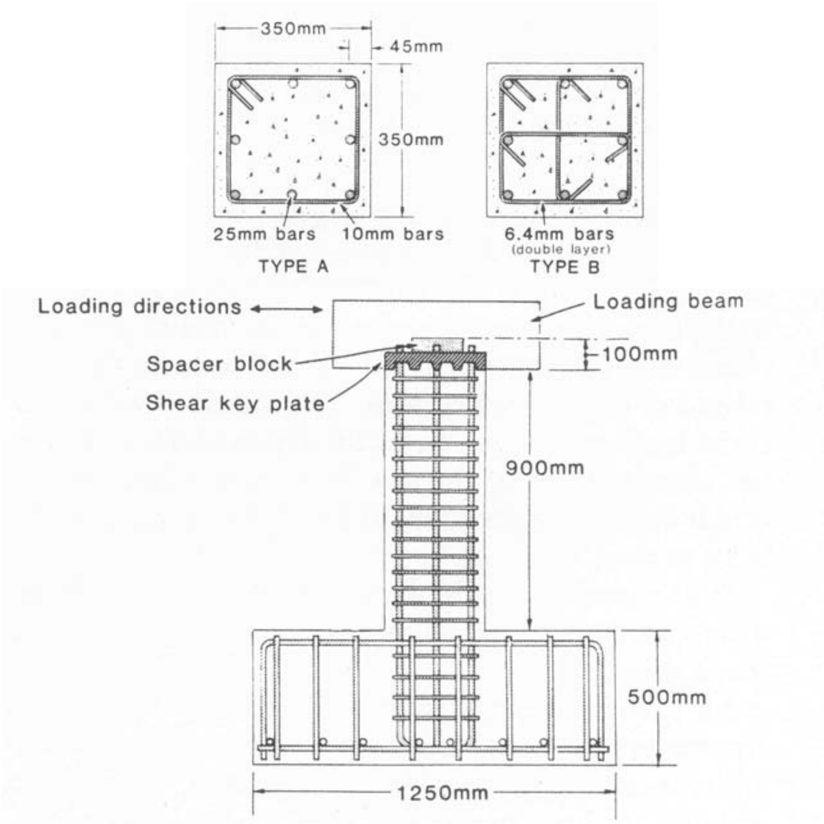


Figure 1.5 - Geometric details of the columns (Ozcebe and Saatcioglu, 1987)

As shown in Figure 1.5 the longitudinal reinforcement arrangement was the same for each specimen. All specimens were designed with excess shear capacities so that the failure would be governed by flexure. The differences between the specimens were in the transverse reinforcement level:

- Type A was used both in U3 and U4 specimens, but in the latter the tie spacing was 67% smaller;
- The lateral reinforcement used in specimen U6 was of the type B. This specimen was designed to have the same shear capacity as the specimen U3 (see last column of Table 1.2), while maintaining approximately the same spacing of transverse reinforcement.

The test setup is illustrated in Figure 1.6. The specimens were subjected to the displacement history shown in Figure 1.7. The quantity Δ_y refers to the yield displacement of the specimen. This parameter was defined as the displacement level at which the critical column section yielded as a whole and was recorded during the test in the region where the rate of strain variation was very high at relatively constant load. All columns were tested under a 600 kN constant compressive axial load, which corresponded to 12% of the nominal column capacity.

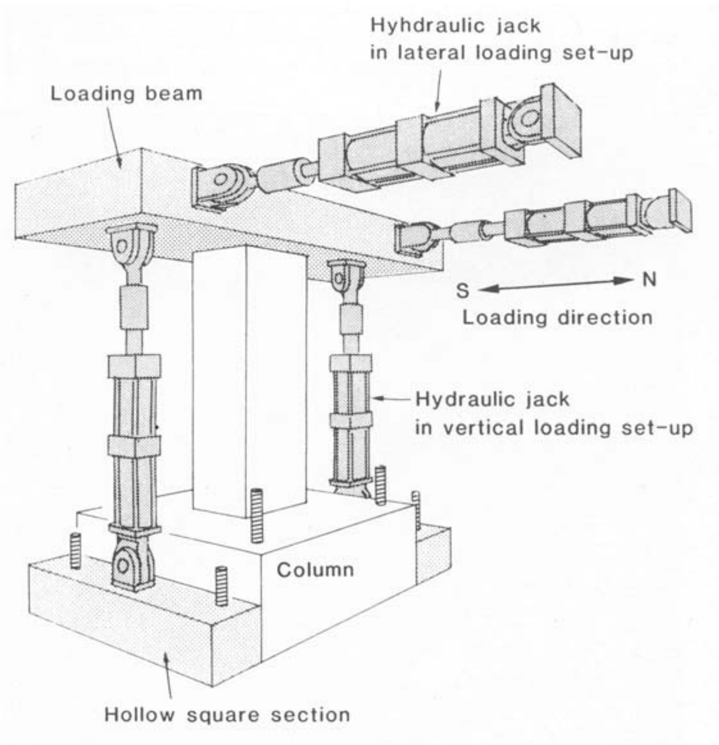


Figure 1.6 - Test setup (Ozcebe and Saatcioglu, 1987)

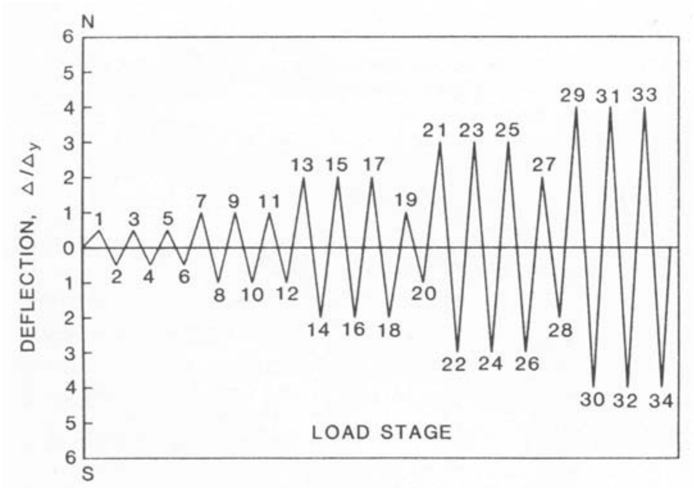


Figure 1.7 - Displacement history (Ozcebe and Saatcioglu, 1987)

In the following the hysteretic force-deformation relationships of the three columns are shown as well the corresponding photographs of the specimens at the end of the $3\Delta_y$ cycles (see Figure 1.8, 1.9 and 1.10).

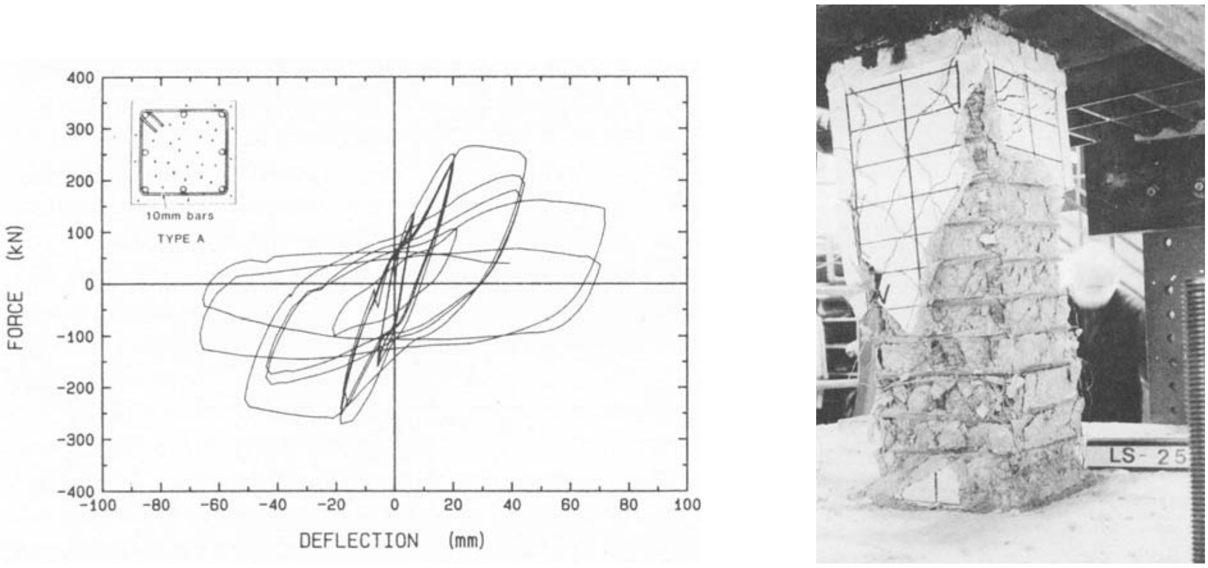


Figure 1.8 - a) Lateral load – top deflection relationship for specimen U3
b) Specimen U3 at the end of $3\Delta_y$ cycles (Ozcebe and Saatcioglu, 1987)

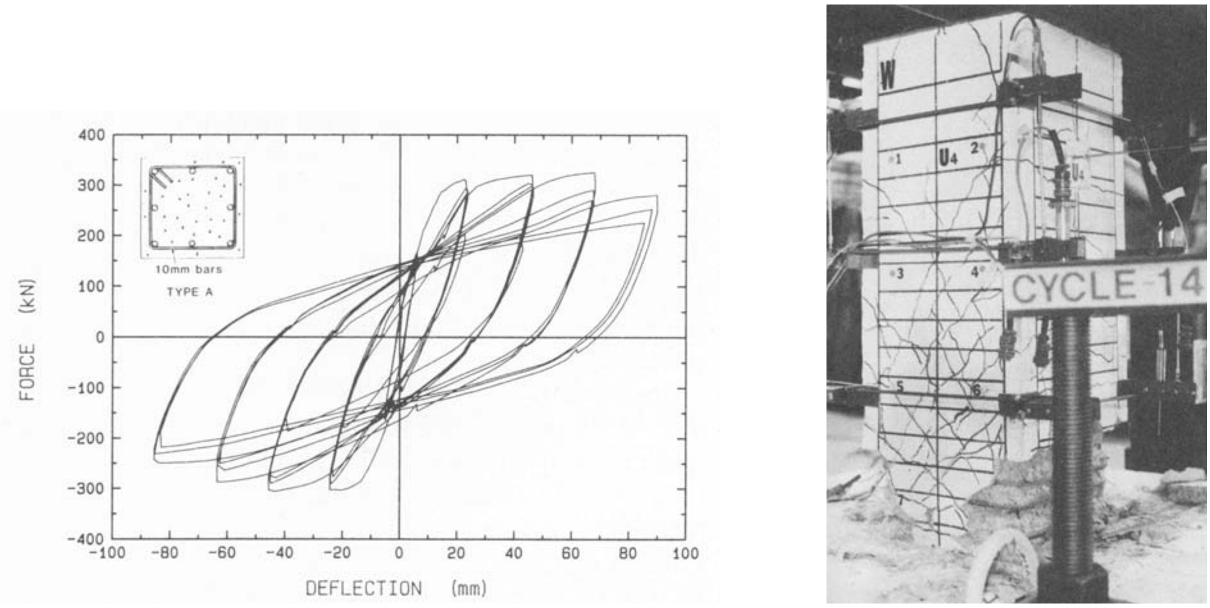
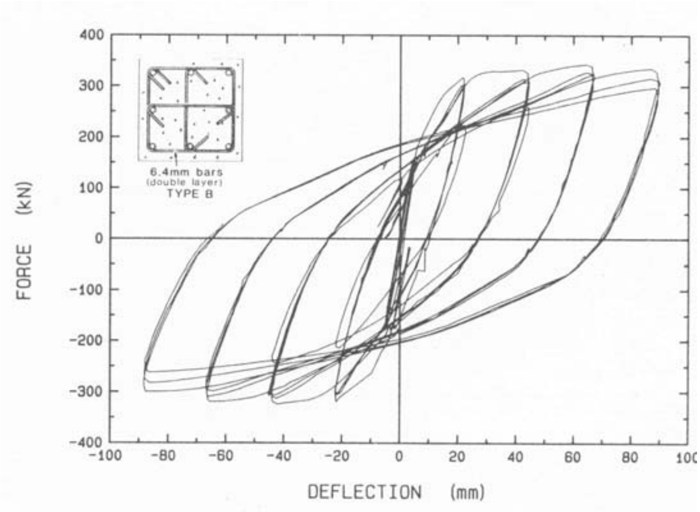


Figure 1.9 - a) Lateral load – top deflection relationship for specimen U4
b) Specimen U4 at the end of $3\Delta_y$ cycles (Ozcebe and Saatcioglu, 1987)



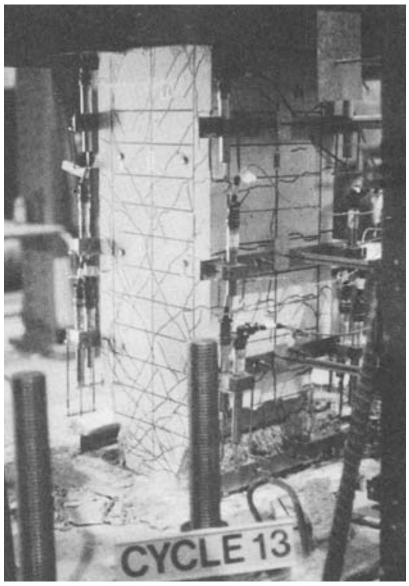


Figure 1.10 - a) Lateral load – top deflection relationship for specimen U6
b) Specimen U6 at the end of $3\Delta_y$ cycles (Ozcebe and Saatcioglu, 1987)

The main conclusions of this experimental work are:

- Comparing the hysteretic force-deformation relationships for specimens U3 and U6 it is evident that, despite having approximately the same amount and spacing of lateral reinforcement, specimen U6 had a superior behaviour with stable hysteretic loops and with negligible strength and stiffness reduction throughout the loading. On the other hand, U3 exhibited a poor behaviour with rapid strength and stiffness degradation. This is confirmed comparing the damage after stage 26 for both specimens (Figures 1.8 b) and 1.10b)). In fact specimen U3 could not survive the cycles at $3\Delta_y$. The explanation for this difference in behaviour is due to the difference in the types of lateral reinforcement. The superiority of Type B configuration lies in the effectiveness of longitudinal column reinforcement in confining the core concrete when supported by the crossties.
- Another parameter investigated experimentally was the influence of spacing and amount of transverse steel. Observing figures 1.8 b) and 1.9 b) we can conclude that specimen U3 experienced significantly larger damage than specimen U4. Referring to the corresponding hysteretic force-deformation relationships it is evident that specimen U4 behaved in a much more ductile manner then U3. In fact, after stage 34 in the loading history, this specimen could still sustain 70% of its peak load. Therefore the behaviour of U4 evidenced the favourable influence of both the amount and spacing of lateral reinforcement. However, comparing the force-deformation relationship of specimen U4 with the one of specimen U6, we can see that the former, despite having a significantly larger lateral reinforcement ratio, ρ_w, had a behaviour close to, but not as favourable as the behaviour of specimen U6. This fact shows that the effect of ρ_w is outweighed by a rational hoop

configuration in what concerns the ductile behaviour of confined concrete. This suggests that a proper choice of confinement is a more feasible solution than increasing ρ_w.

Scott, Park and Priestley, 1982, also studied the effect of spacing of transverse reinforcement on the efficiency of confinement. Their experimental work is also useful for understanding the influence of some basic parameters of confinement. Two main comments can be made:

- Firstly, as expected, both strength and ductility increase with the transverse reinforcement ratio, ρ_w. This is due to the fact that the transverse confining pressure increases with the content of transverse steel.
- The comparison between the behaviour of specimens 18 and 19, in Figure 1.3, can be used to assess the influence of hoop spacing. Both specimens exhibited similar behaviour as the peak stresses were almost the same and the shape of the descending branch was approximately also identical. However specimen 18 had lower transverse reinforcement ratio but the hoops were placed closer than in specimen 19. Therefore one can conclude that similar confinements can be achieved with lower transverse ratios as long as closer spacing is used. The concrete is confined by arching in the concrete between transverse bars (Figure 1.11). If the spacing is large it is evident that a large volume of concrete cannot be confined and may spall away.

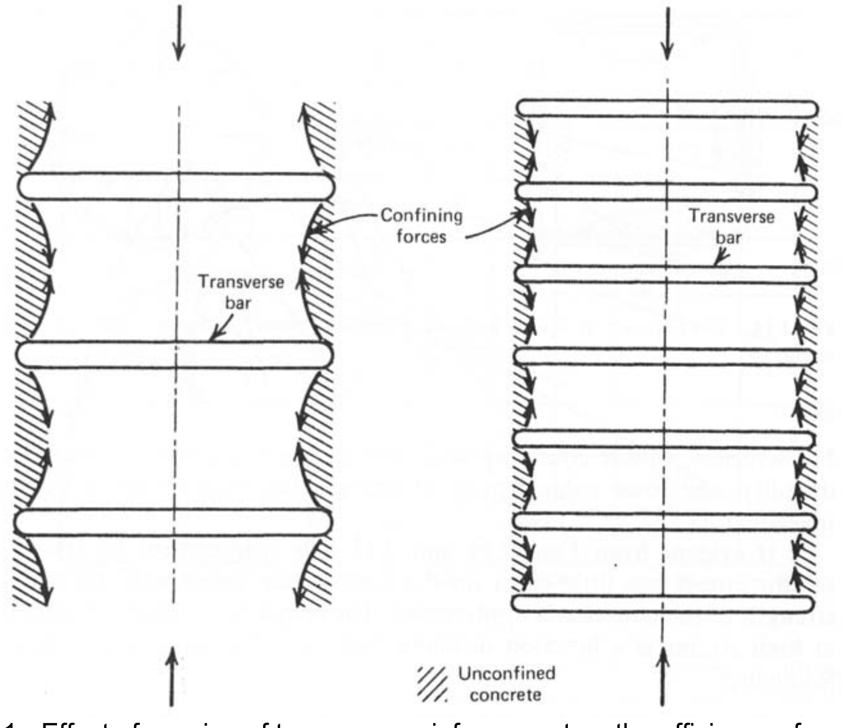


Figure 1.11 - Effect of spacing of transverse reinforcement on the efficiency of confinement (Park and Paulay, 1975)

- The maximum strength of each specimen in the group 12-15 is larger than the corresponding specimen in group 17-20 (see Table 1.1). This indicates better confinement of the specimens with 12 longitudinal steel bars. In fact the closer the reinforcement bars the less is the area of unconfined concrete due to the bending of the sides of the hoops.

In the following the parameters affecting the efficiency of confinement are summarized

i. The yield strength of the transverse reinforcement, because this gives an upper limit to the confining pressure;

ii. The compressive strength of concrete. As discussed before in section 1.1.1. lower strength concrete is more ductile than higher strength concrete. Additionally, the lateral expansion in lower strength concrete is larger, due to the *Poisson effect*, for the same magnitude of axial loading. Therefore the confining pressure is activated sooner in lower strength concrete meaning that the hoops will be more stressed than for higher strength concrete;

iii. The longitudinal reinforcement as concluded from the experimental work carried out by Scott, Park and Priestley (1982);

iv. The ratio of the diameter of the transverse steel to the distance of lateral reinforcement between longitudinal bars. Larger diameters of the transverse steel bars lead to lesser bending and thus to a smaller volume of unconfined concrete along the sides of the hoops;

v. The spacing of the transverse reinforcement as shown in the experimental work carried out by Scott, Park and Priestley (1982);

vi. The transverse reinforced ratio, ρ_w as explained in the work of Scott, Park and Priestley (1982) and Ozcebe and Saatcioglu (1987);

vii. The hoop configuration. See conclusions from the experimental work of Ozcebe and Saatcioglu (1987).

1.2. Reinforcement Steel

1.2.1. Response to monotonic loading

The typical behaviour of steel bars loaded monotonically in tension is presented in Figure 1.12.

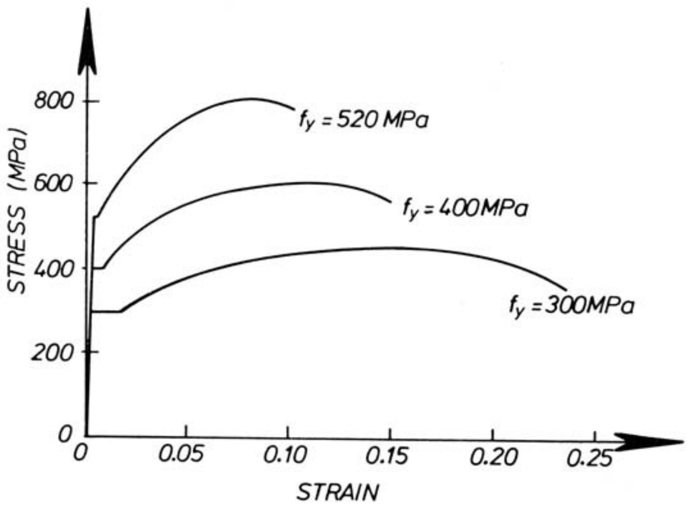


Figure 1.12 - Typical stress-strain curves for steel reinforcement (Paulay and Priestley, 1992)

The main conclusion taken from the results presented in Figure 1.12 is that the behaviour of low-grade steel is more ductile than that of high-grade steel. In fact low--grade steel exhibits a wider and better-defined yield plateau. As the steel grade increases the ratio of peak stress to yield stress increases, which clearly evidences that the influence of strain hardening is larger for high strength steel. Moreover, for high strength steel the ultimate deformation is much less than for low strength steel.

This may lead to the conclusion that the designer interested in seismic protection of the structure would prefer the use of low strength class steel since this is more ductile. However, designing with low strength steel leads to the use of larger diameters of the reinforcement. Larger diameters have unfavourable effects regarding cracking and therefore contribute to a larger strength and stiffness degradation of structural elements (as presented in Chapter 2).

The capacity design procedures used nowadays in most of the relevant code provisions clearly emphasize the importance of a precise evaluation of the strength of each of the structural members so that a desirable energy dissipation system can be formed and maintained throughout the earthquake loading. In practical terms this implies that the strength of the yield zones / plastic hinges should always be lower than all the other sections of structural elements that remain in the elastic range during the earthquake loading.

Therefore neither the actual yield stress of steel should significantly exceed its specified value nor the strain hardening effect should be neglected in the design of the plastic hinges since an increase in the cross-section resistance may affect the strength hierarchy established by the application of the capacity design procedures. Moreover, a structural member more resistant than expected develops higher shear forces than the one estimated during the design phase. Shear forces have a rather unfavourable effect on the seismic behaviour of reinforced concrete members as they drastically reduce its ductility (see section 2.2).

1.2.2. Response to cyclic loading

Experimentally derived curves for steel bars subjected to repeated axial loading (compression or tension) with strain rates, $\dot{\varepsilon}$, similar to earthquake loading with full unloading but no stress reversal, have shown that, as in concrete, the envelope curve practically coincides with the monotonic loading curve. The unloading and consequent reloading branches on the stress-strain diagrams present a narrow hysteretic loop indicating small energy dissipation. In most of the practical idealizations of the steel behaviour under the conditions mentioned above this hysteretic loop is disregarded and so both unloading and reloading branches are assumed to have the same slope as the (first) loading branch, corresponding to the modulus of elasticity, often taken as 200 GPa (CEN 1991).

In the case of seismic loading, depending on the structural element, the reinforcement steel may be submitted to cyclic loading, which implies stress reversals. Figure 1.13 shows the typical stress-strain curve for a steel bar under these conditions.

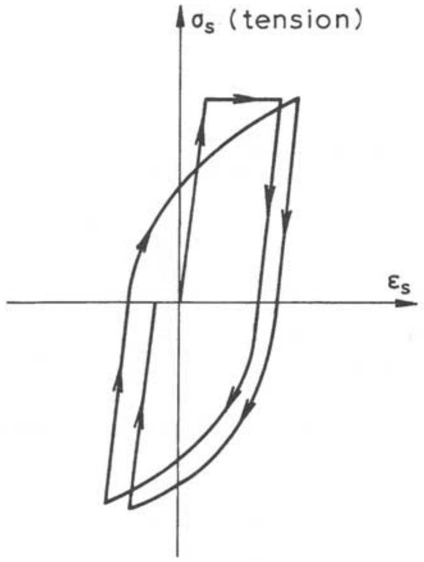


Figure 1.13 - Typical stress-strain curve for steel bar subjected to cyclic loading (Penelis and Kappos)

This figure shows a reduction of stiffness at stresses much lower than the first yield limit after a stress reversal in the inelastic range. This feature is known as the *Bauschinger effect* and it should always be regarded when considering the cyclic behaviour of reinforced concrete. On the other hand, the first part of the unloading takes place almost in an elastic manner and therefore usually its branches are assumed to have the same slope as the first loading branch.

1.3. Bond between concrete and steel

It is widely known that the composite action of concrete and steel is due to bond forces between these two materials. Anchorage may take place along the bars as, in case of plain bars, at its end by means of hooks, anchor plates, etc. To ensure adequate anchorage of the reinforcement steel is the most important aim when detailing reinforced structural elements. Additionally, bond plays a dominant role with respect to seismic behaviour not only because of the reasons above but also because it affects stiffness and energy dissipation capacity.

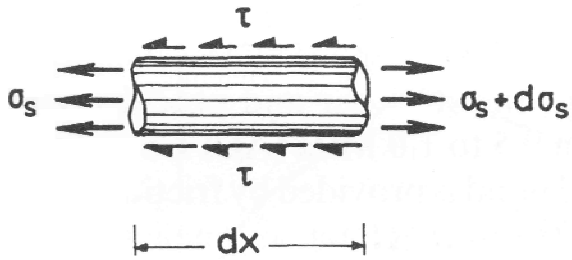


Figure 1.14 - Steel stresses in an infinitesimal element of a plain reinforcement bar (Penelis and Kappos, 1997)

Considering the equilibrium of the forces acting on an infinitesimal element of a plain steel bar, as shown in Figure 1.14, we find:

$$A_s \cdot [(\sigma_s + d\sigma_s) - \sigma_s] = \tau \cdot u \cdot dx \Leftrightarrow$$
$$\Leftrightarrow \frac{d\sigma_s}{dx} = \frac{4}{d_b} \cdot \tau \qquad (1.1)$$

in which τ is the bond stress, u is the bar perimeter and d_b its diameter.

Examining equation (1.1) it appears that bond stresses are zero whenever the steel stress gradient is zero (constant moment areas) whereas its peak value takes place at points of steep gradients (in regions where point loads are applied, for instance).

Figure 1.15 shows the relative slip, s, between the plain bar and the surrounding concrete as a function of the corresponding displacements, u_c for concrete and u_s for steel.

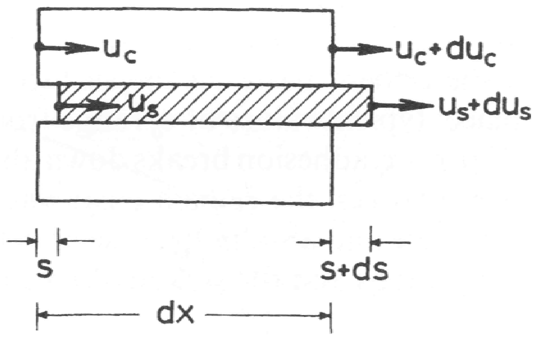


Figure 1.15 - Displacements and relative slip between concrete and a plain steel bar (Penelis and Kappos, 1997)

According to Figure 1.15:

$$\begin{aligned} (s+ds)-s &= [u_s + du_s - (u_c + du_c)] - (u_s - u_c) \Leftrightarrow \\ &\Leftrightarrow ds = du_s - du_c \Leftrightarrow \\ &\Leftrightarrow \frac{ds}{dx} = \varepsilon_s - \varepsilon_c \end{aligned} \tag{1.2}$$

As one may see, the relative slip between the plain bars and the surrounding concrete depends both on the steel strain, ε_S, and the concrete strain, ε_C. However, ε_C is usually disregarded since its value is negligible with respect to ε_S.

Nowadays, the few analytical models for bond are based both on the final results of expressions (1.1) and (1.2) (constitutive equations for bond) and on experimental data. Commonly the bond stress, τ, is expressed as a function of the relative slip, *s.* Penelis and Kappos (1997) provides information about some analytical models for bond. However it should be noted that the results (1.1) and (1.2) apply only to plain bars.

1.3.1. Bond under monotonic loading

The behaviour of bond under monotonic loading is sketched in Figure 1.16. Figure 1.16 shows a qualitative picture of the bond stress-slip relationship. Due to the scarcity of experimental data regarding this subject, consensus has not yet been achieved in the research community regarding quantifying the behaviour of bond under monotonic loading.

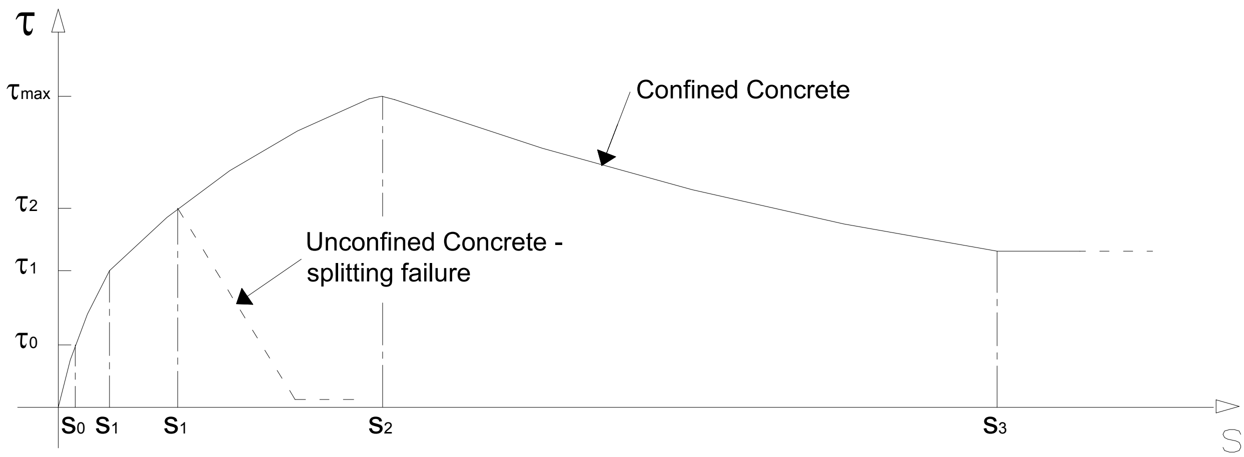


Figure 1.16 - Typical bond stress – slip relationships for unconfined and confined concrete.

Up to a certain level of bond stress, τ_0, almost no slip takes place. In this initial range of stresses, bond is mainly due to chemical adhesion of the cement paste to the surface of the bar. The value of τ_0 ranges from 0.5 to 1.0 MPa for plain bars. When adhesion breaks down, for $\tau>\tau_0$, the bond is assured mainly by friction between the cement past and the microscopic anomalies (pitting) of the bars. For deformed bars, at a bond stress level τ_1, bond cracks form as illustrated in Figure 1.17. Bond cracking is a very complex phenomenon as it depends on several factors such as the strength of the cement paste, the rib spacing and the diameter of the reinforcement bar.

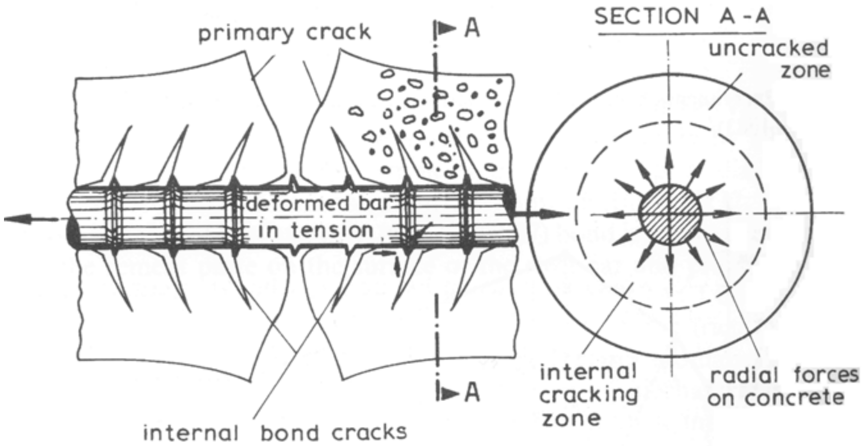


Figure 1.17 - Sketch of the bond cracking mechanism

At approximately the same time as bond cracks form, separation of concrete from the reinforcement bar takes place in the region of primary (flexural) crack. This separation causes transverse displacements leading to an increase in the circumference of the concrete surface previously in contact with the bar and, as a result, circumferential tensile stresses develop (Park and Paulay, 1975). The propagation of bond cracking up to the external face leads to splitting and therefore to the destruction of bond. This happens for levels of bond stresses around τ_2 cf. Figure 1.16. If the reinforced concrete element is not appropriately confined this implies failure (dashed branch in Figure 1.16). Bond stresses along deformed bars are, except for low stresses, due to skew compressive stresses in the concrete. Thus, deformed bars induce transverse displacements in the concrete. Therefore bond strength of deformed bars may be improved by confinement, contrary to plain bars. Confinement inhibits the propagation of the bond cracking, mainly due to the fact that transverse compression is beneficial to the anchorage of the reinforcement bar. Therefore, for confined elements the bond resistance can reach significantly higher values (τ_{max}). Additionally, the presence of confinement leads to a more ductile behaviour as it inhibits the bond failure due to splitting. After the maximum bond stress, τ_2, a progressive deterioration of the concrete between adjacent ribs takes place (descending branch, in Figure 1.16). The following moderate residual bond stress takes place for values of slip around s_3, due to friction at the cylindrical surface defined at the tips of the ribs.

1.3.2. Bond under cyclic loading

As for the monotonic loading, the bond resistance under cyclic loading is still only qualitatively understood. In the following, a brief description of the most important features governing the cyclic behaviour of bond will be discussed.

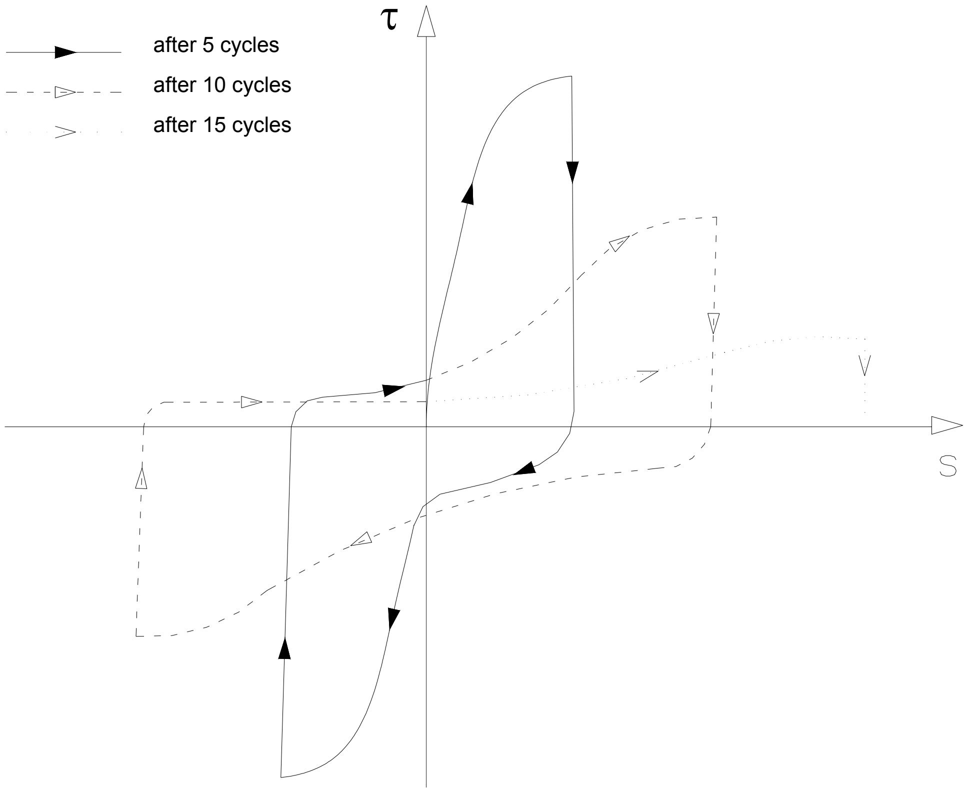


Figure 1.18 - Typical bond stress-slip relationship of a deformed bar under cyclic loading

Observing Figure 1.18 three main features can be pointed out to describe the behaviour of bond under cyclic loading:

- The residual slip during unloading is quite large. This is mainly due to the fact that the elastic part of slip consists in the concrete deformation only, which is negligible regarding the contribution of the steel deformation. Also microcracking in the concrete and the release of shrinkage strains result in some permanent slip. Therefore the cracks formed during the tensioning of a bar cannot close completely by removal of the load.

- One can distinguish two different parts in the reloading branches: The first part with relatively small slope up to slip values around the ones achieved in the previous cycle; The second part with a higher slope for slip values larger than in the previous cycle. It can be easily understood that the bond resistance in the first part is merely due to friction between the bars and the concrete surface around. For the second part the reinforcement bar comes in contact with intact concrete leading to an increase of the stiffness.

- Large softening effect and fast strength decay. Figure 1.18 shows clearly the slope reduction of the reloading branches after each cycle. This has to do with the gradual smoothening of crack interfaces, which causes a reduction of the mechanical interlock and friction forces (Penelis and Kappos, 1997).

2. Inelastic Response of Reinforced Concrete Elements in Cyclic Loading

The following chapter presents an overview of the behaviour of reinforced concrete members in cyclic loading. As mentioned before, emphasis will be on analysis rather than on design. Therefore, firstly, the discussion will be held in terms of the response of reinforced concrete members with respect to cyclic actions, instead of presenting the cyclic behaviour of members regarding their structural function. As in the previous chapter relevant experimental work will be described to provide and illustrate a clear understanding of the basic characteristics of strength and ductility throughout seismic loading. The last section of this chapter discusses the cyclic behaviour of joints in a qualitative manner. The reason for this approach lies in the lack of consensus in the main schools of thought and to the scarcity of experimental data available.

2.1. Members with flexure-dominated behaviour

2.1.1. Members in uniaxial flexure

Most of the experimental work done to date about the cyclic behaviour of reinforced concrete members has dealt with the simplest case of uniaxial flexure under zero axial force. Although even for beams this case seldom occurs during a seismic action, its discussion is considered to be of high value to understand the cyclic behaviour of reinforced concrete members in which the flexural mode of behaviour dominates over the shear mode.

2.1.1.1. Members with symmetric cross-section and reinforcement

Experimental work carried out by Brown and Jirsa (1971)

Brown and Jirsa (1971) carried out an experimental work to determine the effect of load history on the strength and ductility and mode of failure of cantilever beams. For the present purpose only the results with respect to specimens subjected to reversed loading histories are going to be discussed.

The tests specimens were cantilever beams cast with an enlarged end block. The load was applied in the free end of the beam. The cross-section was rectangular 15.2x30.5 cm^2. The specimen designation (Figure 2.1) contains information about the reinforcement used, the load and the length of the shear span: each of the numbers in the first pair of numbers stands for the amount of longitudinal reinforcement bars used in the top and bottom face, respectively (the same diameter was used for the longitudinal bars in all the specimens); the second pair of numbers refers to the web reinforcement: the first number has to do with the amount of stirrups and the second one with their spacing in inches (1 in.=2.54 cm); the designation RV5 or RV10 means that the load applied is reversed and the deflection limit is 5 or 10 times the yield deflection; the last number represents the length of the shear span, also in inches.

The test results are illustrated in Figure 2.1 in the form of diagrams of force-displacement in terms of loads (*F*) versus deflections (δ). Diagrams of this type together with the ones depicting moments (*M*) versus rotations (θ) are the best to illustrate the response of a reinforced concrete member under cyclic loading.

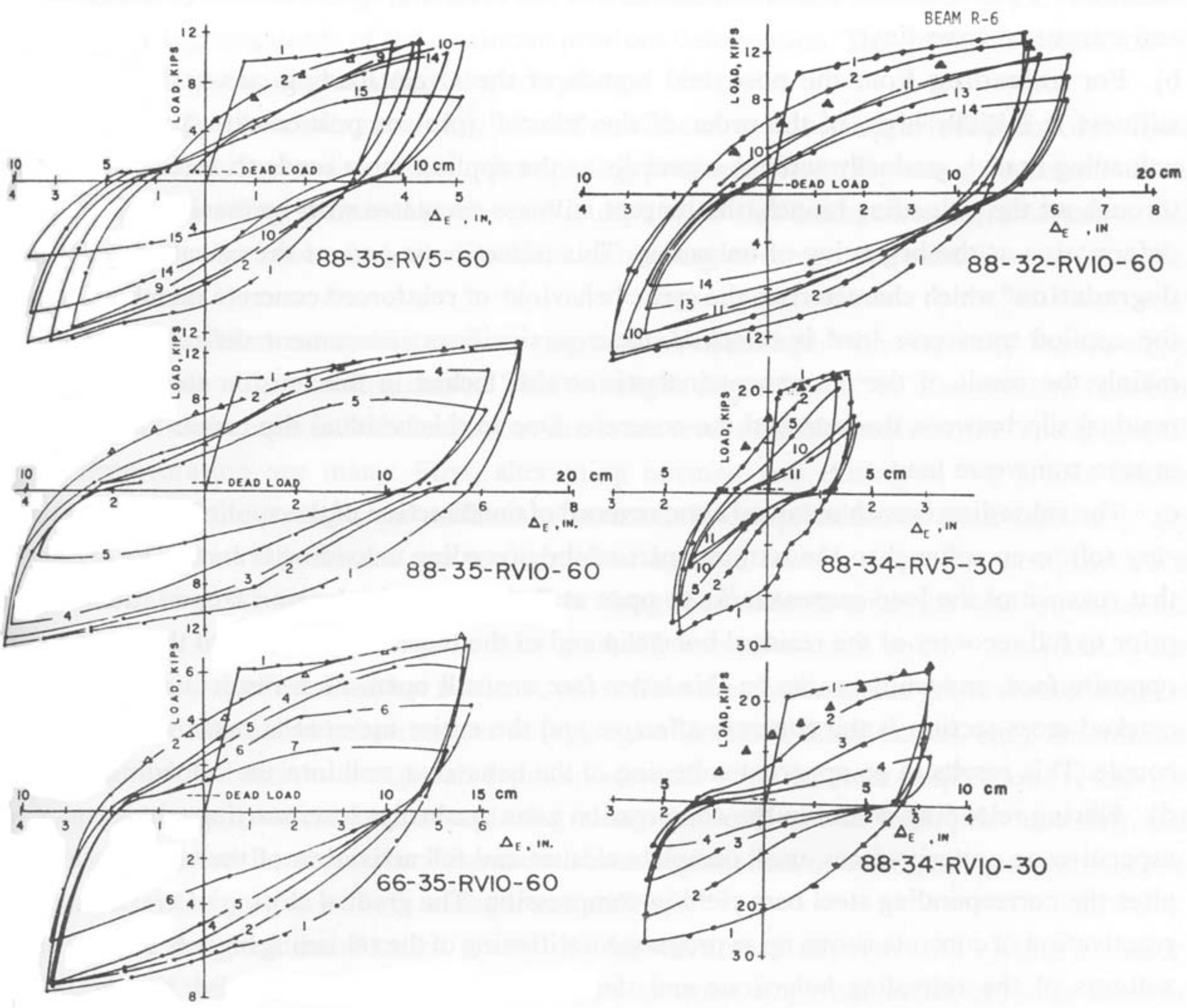


Figure 2.1 - Representative force-deflection loops of specimens with symmetric cross-section and reinforcement in cyclic uniaxial flexure with zero axial load (Brown and Jirsa, 1971)
(1 kip = 4.45kN;1in. = 2.54 cm)

The main features of the curves in Figure 2.1 are:

- The stiffness gradually deteriorates in the first loading branch. This is particularly evident in specimen 88-32-RV10-60. In fact, as the load increases flexural cracks develop in the tensile face and after a certain limit bond slip between the reinforcement and the surrounding concrete takes place (*Zone **a)*** in Figure 2.2). The following abrupt softening of the response is mainly due to yielding of the tension steel at the cross-section of maximum moment (*Point **b)*** in Figure 2.2). After yielding the resistance of the member keeps increasing although its stiffness is much more reduced. This has to do with the reduction of the neutral axis depth due to the large post-yield extension of the tension steel, increasing the lever arm of the internal forces. Also strain hardening of the tension steel (see section 1.2) contributes to a positive slope of the post yield branch for the first loading (*Branch **c)*** in Figure 2.2).

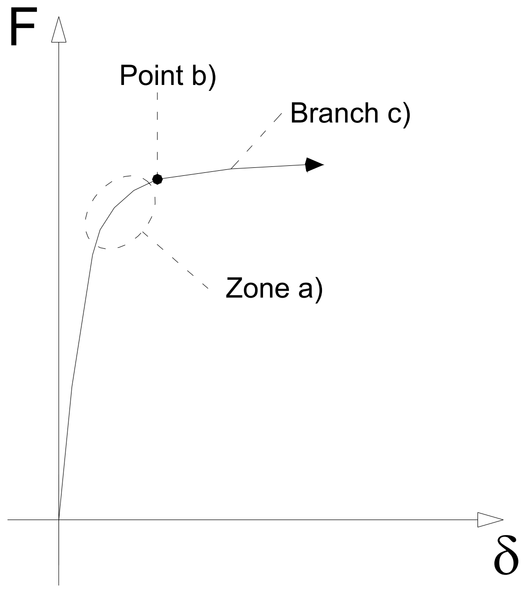


Figure 2.2 - Sketch of the first loading branch for a symmetric R/C member

- The initial stiffness of the unloading branches is high, of the order of the elastic stiffness *(Zone **d)*** in Figure 2.3) and then gradually softens as the applied load tends to zero *(Branch **e)*** in Figure 2.3*)*. When the load is removed there will be significant permanent deflections due to inelastic strains locked in the tension steel previously in the plastic domain and to the residual slip between the reinforcement and the concrete. This is the reason for the cracks to remain open even at zero load *(point **f)*** in Figure 2.3).

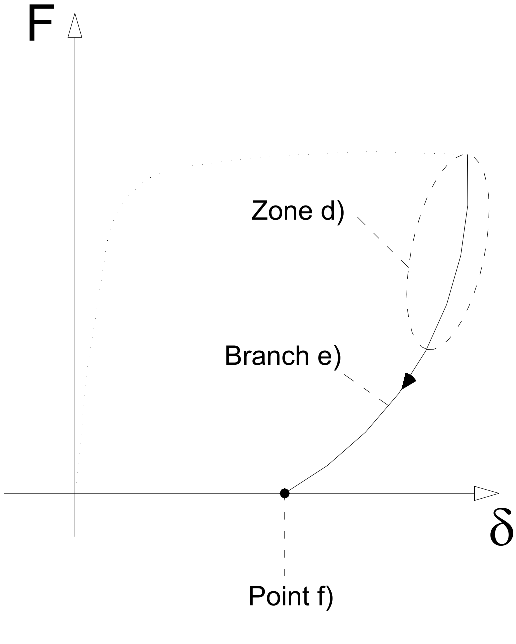


Figure 2.3 - Sketch of the first unloading branch for a symmetric R/C member

- The initial part of the reloading branches in the opposite direction is rather flat. In fact even more than the terminal part of the preceding unloading branch. The reason is that the cracks on the face previously in tension are still open as explained previously and when reloading in the opposite direction takes place, cracks will open on the new face under tension. This will take place before the full recover of the residual bond slip and of the inelastic extension of the bars on the opposite face and therefore before the closing of the cracks there. As a result the whole cross-section is cracked and the concrete is ineffective, which will lead to the entire moment being resisted by the steel couple alone *(Branch **g**)* in Figure 2.4). As the magnitude of reloading increases, the cracks in the new face under compression gradually start to close after the corresponding steel bars yield in compression. This means the reactivation of the concrete and consequent stiffening of the reloading branch. *(Zone **h**)* in Figure 2.4). The succession of the softening-stiffening effect in the reloading branches moves the curve towards the origin, as this would be "pinched". For this reason this effect is commonly designated as *pinching*. This is a very important feature to be considered when analysing the energy dissipation capacity of a structural member. The more pronounced pinching is the less effective is the member in absorbing the energy induced from cyclic loading.

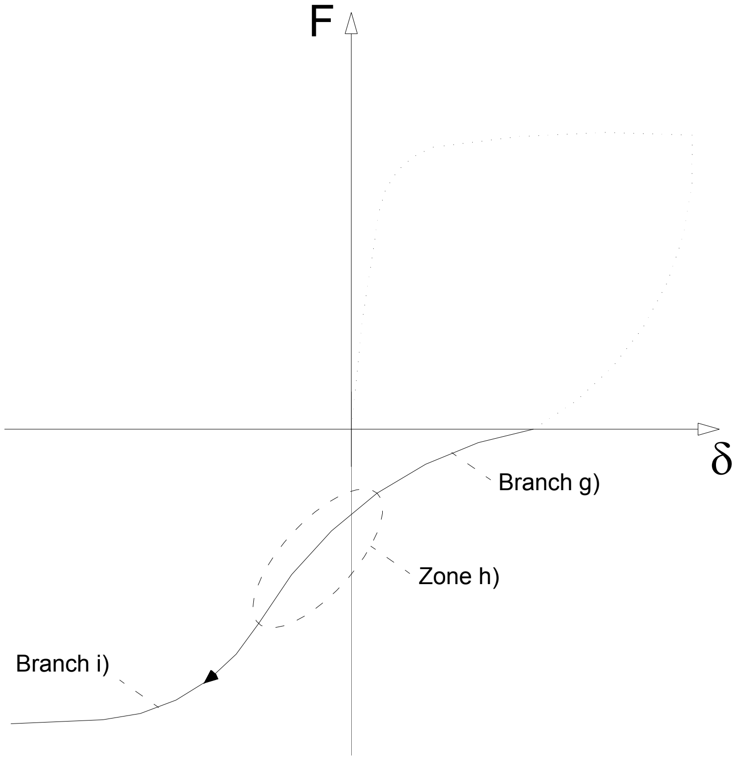


Figure 2.4 - Sketch of the first reloading branch on the opposite direction for a R/C member with symmetric cross-section and reinforcement in uniaxial flexure.

- After the stiffening that terminates the pinching effect in the reloading branch, a second gradual softening again can be observed *(Branch **i**)* in Figure 2.4). This is the *Bauschinger effect*, mentioned in section 1.2.2, affecting the steel bars. In fact, the steel bars now in tension have yielded in compression during the previous half cycle and vice-versa for the bars now in compression. Therefore the steel bars start to yield earlier than for the first loading branch, meaning earlier softening in the reloading branch.

- The following unloading-reloading cycles follow the same pattern as described above. However the reloading branches seem to approach the point of extreme deformation in smoother lines as the number of cycles increases *(Branch **j**)* in Figure 2.5). This is the so-called *stiffness and strength degradation process due to cyclic loading*.

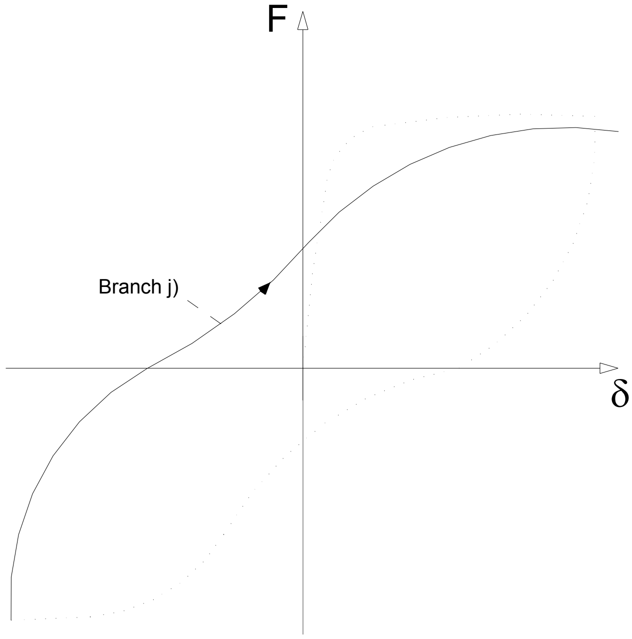


Figure 2.5 - Typical branch of unloading-reloading cycle for a symmetric R/C member

One of the main reasons for the degradation of the structural properties of the member is the gradual increase of the influence of shear deformations. Comparing Figure 2.6, which shows the end rotations of the beam for specimens 88-35-RV10-60 and 88-34-RV10-30 with the corresponding force-deflection curves (Figure 2.1), it is evident that despite the peak deflections roughly remain constant the end-rotations are reduced significantly, meaning an increase in the magnitude of shear deformations. This together with the alternate opening and closing of the cracks cause a degradation of the concrete stiffness and strength in compression, as crack faces may not come into full contact. Another important factor contributing for the stiffness and strength degradation of the member is the bond deterioration mechanism with cycling as explained in section 1.3. The bond between concrete and steel gradually becomes less effective, which increases cracks widths, contributing to larger *pinching* and reduces the tension-stiffening effect. Also the combined effect of the whole cross-section being ineffective, so that the moment applied is resisted by the steel couple alone, with shear increases the splitting of the concrete along the longitudinal bars. This leads to further bond deterioration and in certain cases may cause the spalling of the concrete cover by dowel action.

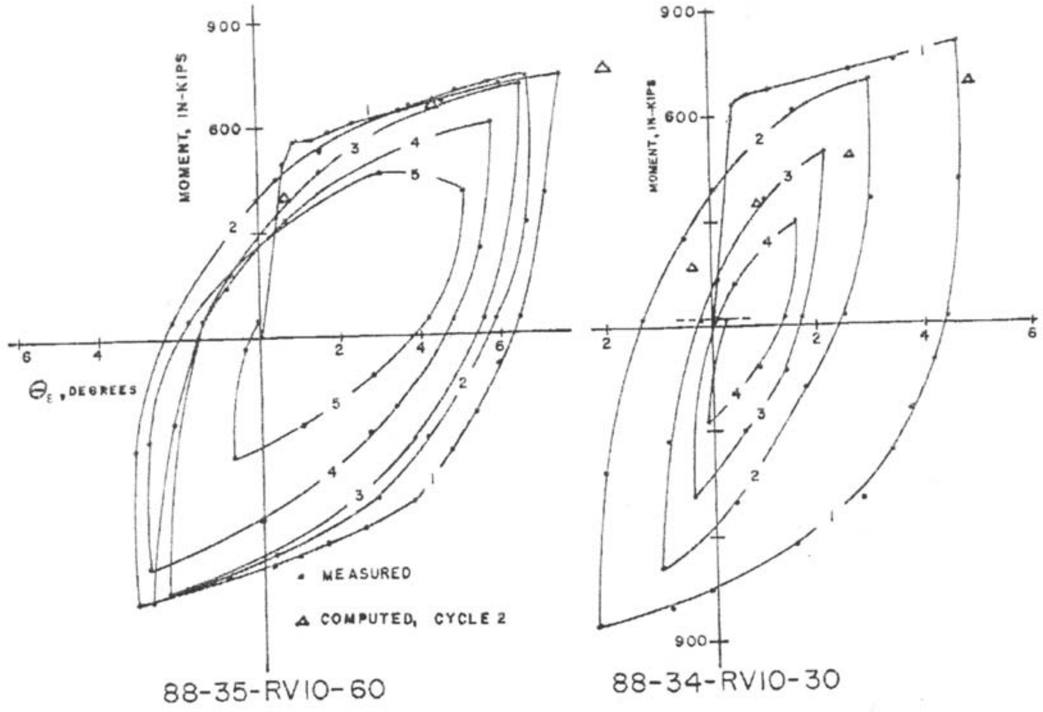


Figure 2.6 - Load-end rotation curves for specimens 88-35-RV10-60 and 88-34-RV10-30 (Brown and Jirsa, 1971) (1 kip =4.45 kN)

The experimental work carried out by Brown and Jirsa is also useful to illustrate two important parameters influencing the cyclic response of structural members:

- As the *longitudinal steel ratio* increases, the larger the stiffness and strength degradation the less is the energy dissipation capacity. This is evident since specimens reinforced with 8 bars failed in fewer cycles than did those reinforced with 6 bars. In fact as the load capacity increases the greater is the shear on the cross-section and so the larger are the shear deformations with the consequent deterioration of the structural properties of the member. Moreover an increase of the flexural reinforcement ratio increases the compressive stresses of the concrete, and thus increases the rate of degradation.
- Reduction of the *stirrup spacing* significantly increased the number of cycles, as can be easily observed by comparing the force-deflection curves for specimens 88-32-RV10-60 and 88-35-RV10-60 in Figure 2.1. It is also evident that the response of the specimens with closer spacing is superior in leading to more stable hysteresis loops. This is due to the fact that better confinement of the concrete core is achieved with closer spacing of the transverse reinforcement as explained in section 1.1.2. As mentioned there adequate confinement has the favourable effect of keeping the inner structure of the concrete member preserved diminishing the damage due to sliding along the cracks from shear deformations.

Finally some remarks should be made regarding the *failure mode* in structural members with symmetric cross-section and reinforcement under uniaxial flexure. Generally in these cases, failure is caused by progressive deterioration of the compressive zones of the concrete combined with the growing influence of the shear deformations. If the member is submitted to a strong imposed displacement history, i.e. with peak displacements several times larger than the yield displacement as the specimens in the tests of Brown and Jirsa (1971), the damage is first observed in the concrete on the faces of the cross-section with the highest bending moment. This damage is due to successive states of high compression, leading to crushing of the cover concrete. This effect, together with the increase of bond slip between the steel bars and the surrounding concrete, leads to the separation (spalling) of the concrete cover exposing the steel bars. At this stage the bar may buckle due to loss of lateral support (Figures 2.7 a) and c)). This type of failure is characterized by a high value of the ductility factor and significant energy dissipation during cyclic loading (Penelis and Kappos, 1997). In Figure 2.7 three different modes of buckling of the longitudinal reinforcement are illustrated.

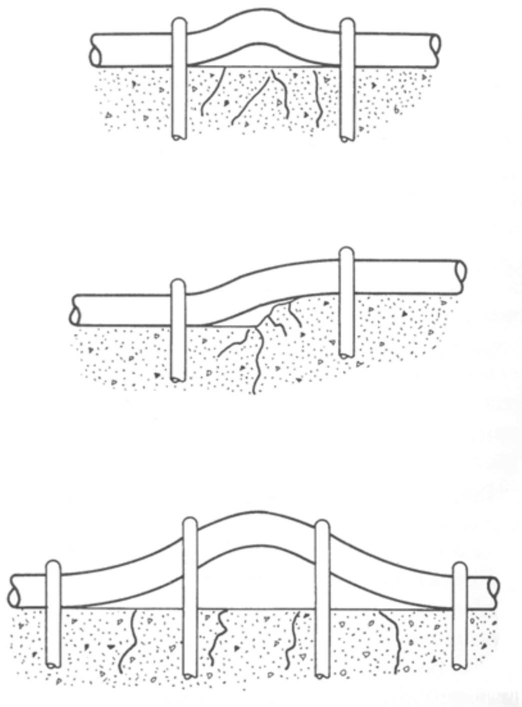


Figure 2.7 - Different modes of buckling of reinforcement bars (Penelis and Kappos, 1997)

Figure 2.7 shows the importance of the transverse reinforcement spacing in preventing buckling of the steel bars: the closer the stirrups the better they can provide lateral support to the steel bars after spalling of the concrete cover due to reduction in the buckling length of the bar (Figure 2.7 a)). Also the yield strength of the steel used in transverse reinforcement should not be too low in order to prevent buckling of the longitudinal bars as shown in Figure 2.7 c).

Another mode of failure is the one dominated by shear deformations. This is characterized by rapid deterioration of stiffness and strength (brittle failure). As the shear deformations increase with cycling, the abraded surfaces of the open flexural vertical cracks lose their shear capacity due to cyclic sliding. The propagation of the diagonal shear cracks leads to the progressive degradation of the compressive strength of the concrete core, which is evidenced by a rapid stiffness and strength decay in the force-deflection curves (see in Figure 2.1 the curves referring to specimens 88-34-RV5-30 and 88-34-RV10-30). Significant shear deformations may also cause lateral buckling of the longitudinal bars as shown in figure 2.7 b). This mode of failure will be discussed in more detail in section 2.2.

Wight and Sozen (1973) undertook a test series to investigate the mode of failure for reinforced concrete columns subjected to several load reversals and to deflections larger than the yield deflection. Figure 2.8 illustrates the development of the crack pattern in a cantilever specimen to a high level of damage.

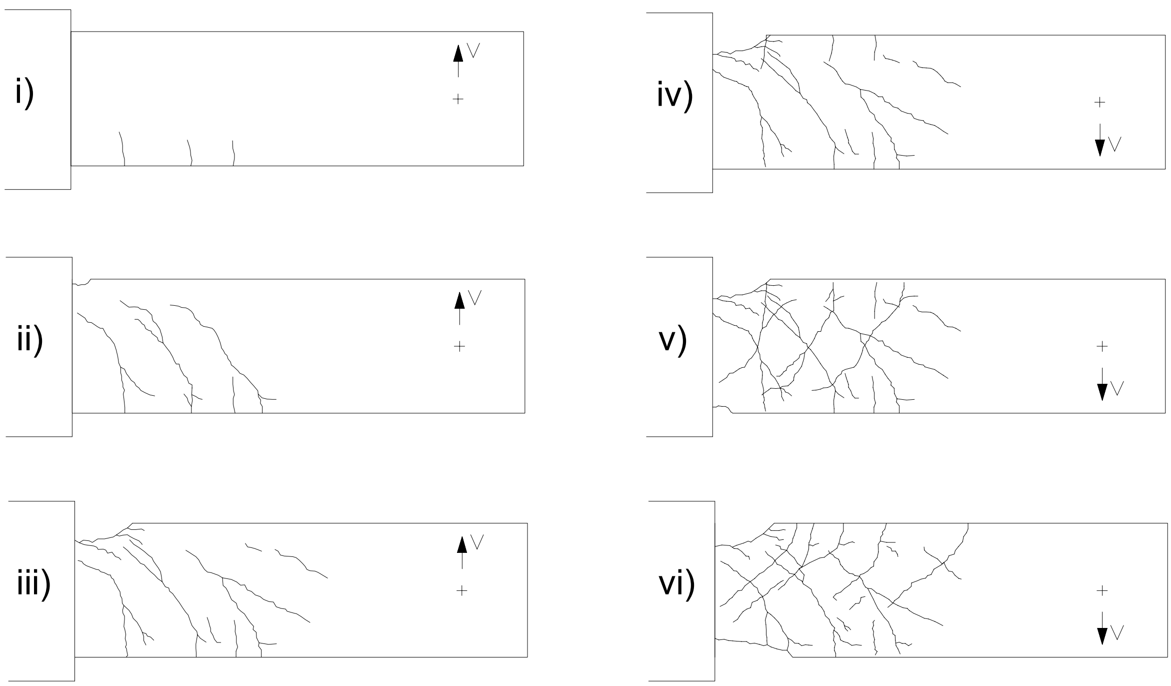


Figure 2.8 - Development of a crack pattern (Wight and Sozen, 1973)

It appears that the first cracks occur at the tensile face of the cross-section with maximum moment (in this case, the support cross-section). As the deflection and load continue to increase, inclined cracks emerge from the vertical cracks and splitting cracks form along the tensile reinforcement. In the figures the development of spalling of the cover concrete in the compressed zone with the succession of cycles of deflections beyond the yield limit may be observed. The reversion of the load leads to a greater damage of the concrete member as new cracks intercept the ones formed in the previous half cycle.

These tests led to the following observations regarding the influence of the transverse reinforcement ratio, ρ_w, on the pattern of failure mode in concrete members. As this parameter increases:

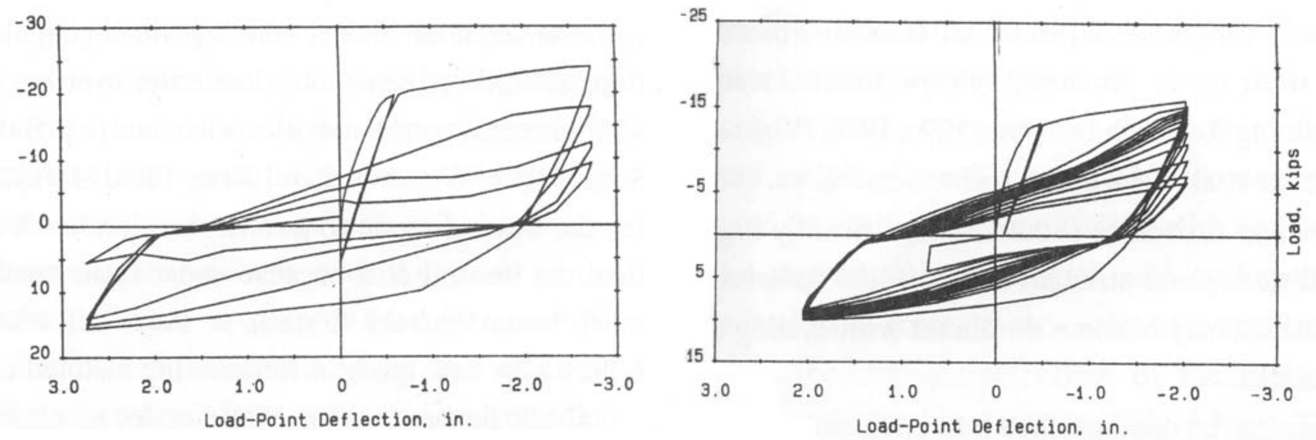
- The region of inelastic behaviour (plastic hinge) extends over a smaller area;
- The extension of the spalling of the cover concrete and the splitting in the tensile face diminish.

Therefore it may be concluded that transverse reinforcement can be used to control the extent of damage along a reinforced concrete member designed to undergo inelastic deformations.

2.1.1.2. Members with asymmetric cross-section and/or reinforcement

Most of the structural members used to withstand flexure are beams. Usually these members are not designed with symmetric cross-section and/or reinforcement. Further, the shape of the cross-section is not rectangular, but T or L and the amount of top steel used is different from the amount of steel used in the bottom. Also in "symmetric" beams monolithically cast with a slab the combined action of the slab and beam leads to a larger effective flange. Therefore asymmetric behaviour should be considered.

Nmai and Darwin (1986) carried out an experimental work on lightly reinforced concrete beams under cyclic loading, in which asymmetric specimens were tested. The members had rectangular cross-section (190 x 457 mm^2). In the specimens F2 and F4 the top reinforcement consisted of 6 and 4 bars of 13 mm of diameter, respectively. The same bars were used for the bottom reinforcement: 3 bars for specimen F2 and 2 bars for specimen F4. The specimens were submitted to deflection amplitudes of 5 times the yield deflection. The results of the tests are depicted in Figure 2.9



F2 ***F4***

Figure 2.9 - Load-deflection curve for specimens F2 and F4 (1 kip =4.45 kN; 1in. = 2.54 cm)

The main difference in the cyclic flexural behaviour between members with symmetric cross-section and reinforcement and those with asymmetric cross-section and/or reinforcement is that the latter exhibit asymmetric hysteresis loops (CEB, 1994) – Figure 2.9. This difference is due to the fact that the stiffness and strength of the member is not the same for both loading directions. For the present case the amount of bottom reinforcement is lower than the amount of top reinforcement. Therefore the strength and stiffness of both sections to positive moments are lower than for negative moments.

As it can be seen in Figure 2.9 the specimens were first loaded in their “strong” direction (bottom reinforcement in compression and top reinforcement in tension). The abrupt decay in stiffness after the “elastic” branch indicates yielding of the top reinforcement due to tension. When reloading in the “weak” direction (bottom reinforcement in tension and top reinforcement in compression) takes place, the cracks in the face previously in tension are fully open as in the case of symmetric cross-section (see in the previous section the explanation given about the *branch g)* in Figure 2.4). The difference now is that those cracks remain open throughout the entire reloading in the “weak” direction. This is because tensile yielding of the bottom reinforcement is not sufficient to cause yielding of the top reinforcement due to compression. So, as long as yielding in the strong direction has taken place, reloading in the “weak” direction is characterized by full-depth open cracks. Therefore when the member is loaded in the “weak direction”, the steel couple alone resists the moment and the gradual stiffening caused by closing of the cracks does not take place. Thus, the reloading branch in the “weak” direction is of very low stiffness and without pinching. Pinching does take place upon reloading in the “strong” direction and is rather pronounced. This is due to the fact that only in the reloading branch for the “strong” direction, the concrete starts to become effective again and the top reinforcement is still elastic, as it did not yield in compression in the previous half-cycle.

For this kind of members, failure usually develops in two ways: Failure in the “strong” side in tension or failure in the “weak” side in tension. The first one has a gradual development characterized by progressive disintegration and crushing of the concrete in the “weak” side. This failure mode is the most desirable one as it may reach high levels of ductility. The second mode of failure is rather brittle as it involves the fracturing in tension of the steel bars in the “weak side” and therefore a sudden drop in strength. Often this happens after the steel bars have buckled due to compression in the previous cycles.

2.1.2. The effect of axial forces

In the following the influence of axial loading on the cyclic behaviour of flexure-dominated reinforced concrete members is discussed. Most of the structural members in which the effect of axial loading in seismic performance has to be considered are columns. The way in which the degradation of the response of these members due to reversed cyclic loading develops is similar to that described in section 2.1.1. However, most of the times these elements are subjected to a biaxial state of flexure. Nevertheless, it is found appropriate to discuss the uniaxial flexure case as this allows a better understanding of the influence of the axial loading by comparison with what discussed in the previous section. Also, the way in which the axial forces influence the cyclic responses of members is similar both in uniaxial and biaxial flexure.

Experimental work carried out by Saatcioglu and Ozcebe, 1989

The authors mentioned above carried out a test series with the intention of investigating the response of reinforced concrete columns to seismic loading. The experimental program is quite similar to the one referred to in section 1.1.2.. In fact, the specimens tested had the same geometry and were reinforced both longitudinally and transversely as in the 1987 tests (Figure 1.5). The test setup is the same as shown Figure 1.6.

The differences were in the loading program: Three different groups of specimens were tested, each one labelled according to the deformation path imposed. Specimens U were loaded uniaxially in the direction parallel to the principal axis of the column with the same loading history as shown in Figure 1.7; Specimens D were loaded as specimens U but the load was applied along the diagonal of the section; Specimen B1 was subjected to a bi-directional deformation path as shown in Figure 2.10. This loading history was intended to simulate a major seismic action in one direction while a minor action was occurring in the orthogonal direction. The specimen was designed to have the same capacity in both directions. The properties of the different columns are shown in Table 2.1.

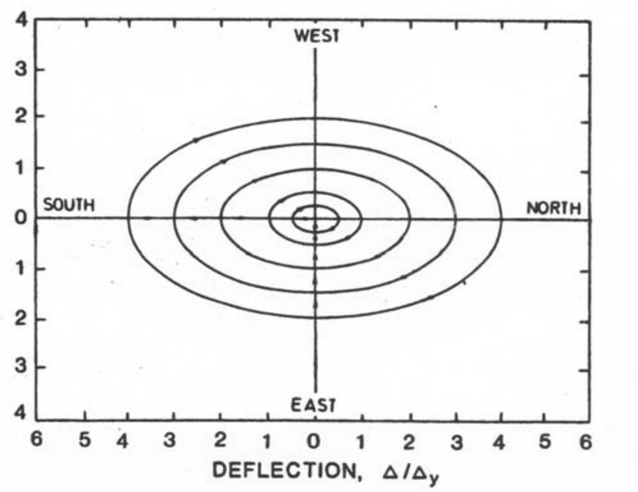


Figure 2.10 - Loading history for specimens B1 (Δ_y = yield displacement of the specimen) (Saatcioglu and Ozcebe, 1989)

Table 2.1 - Properties of the columns (Ozcebe and Saatcioglu, 1989)

Test Specimen	*Concrete Strength (MPa)*	*Longitudinal Steel*	*Transverse Steel*				Axial Load (kN)
		$f_{y,l}$ *(MPa)*	$f_{y,t}$ *(MPa)*	ρ_w ($^o/_{oo}$)	*s, mm*	*Configuration*	
U1	43.6	430	470	0.85	150	Type A	0
U2	30.2	453	470	0.85	150	Type A	600
U4	32.0	438	470	2.54	50	Type A	600
U6	37.3	437	425	1.95	65	Type B	600
D1	40.3	453	470	0.85	150	Type A	0
D2	30.2	453	470	0.85	150	Type A	600
D4	43.6	430	470	2.54	50	Type A	600
B1	32.0	438	470	2.54	50	Type A	600

2.1.2.1. Members with constant compressive axial force

Depending on the intensity, compressive axial loading may have favourable as well as unfavourable effects on the ductility, strength and stiffness degradation throughout the seismic response of a structural member.

The presence of compressive axial stresses contributes to the closing of the flexural cracks. This is reflected in the final phase of unloading from a post-yield peak displacement and the first stage of reloading in the opposite direction. The additional compression state of stress due to axial loading accelerates the yielding in compression of the steel bars that have previously yielded in tension and are now going to compression. As a result nowhere during the loading cycle the cracks are open through the full depth of the cross-section and therefore the steel couple never resists the moment alone. This means the “suppression” of *Branch g)* (see Figure 2.4) in the first part of the reloading branch in the opposite direction. Thus the *pinching* effect, typical of the cyclic response of the structural members dominated by flexure, is not observed in a pronounced way. This means an improvement in what concerns the energy dissipation capacity.

Also the stiffness of the virgin loading, the unloading and reloading branches increase. This is mainly due to the increase of the depth of the compressed concrete and hence an increase of the contribution of the concrete for the overall stiffness.

As for the flexural cracks, compressive stresses also contribute to the closing of the cracks perpendicular to the axis of the member diminishing the risk of premature failure due to sliding shear.

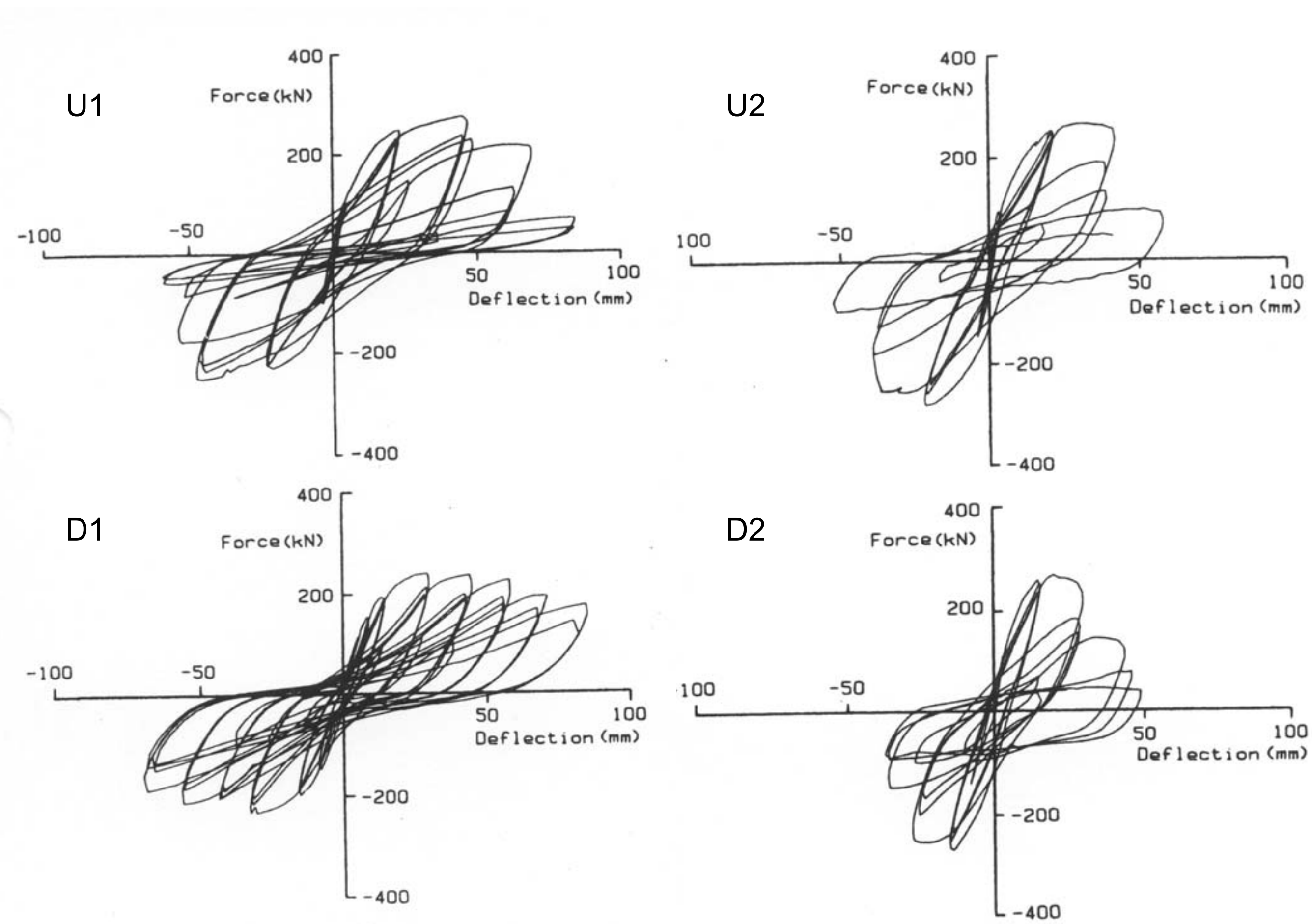


Figure 2.11 - Test results for specimens U1, U2, D1 and D2 (Saatcioglu and Ozcebe, 1989)
The hysteretic relationships are in terms of lateral load – top deflection in the loading direction

In contrast with these favourable effects it should be noticed that from a certain level of compression the consequences are severe regarding ductility reduction and acceleration of failure.

- The presence of compressive stresses results in a larger compression zone in the cross-section and thus in higher demands regarding concrete strains. Thus, crushing and degradation of the concrete core combined with spalling of the concrete cover take place at lower levels of displacement with subsequent drop of strength. This can be easily observed comparing the results from specimens U1 with U2 and D1 with D2 in Figure 2.11. Members with high concrete covers due to environmental conditions are particularly exposed to this effect. As a consequence the longitudinal reinforcement is exposed faster and therefore the risk of buckling due to compression is higher and may develop sooner. This last effect is actually the cause of the most common type of failure of columns subjected to high levels of axial load. This is the reason why most of the modern code provisions clearly emphasize the importance of adequate transverse reinforcement as explained in connection with Figure 2.7. in section 2.1.1.1. Moreover proper transverse reinforcement (with adequate detailing and close spacing of the stirrups) improves confinement of the core concrete and therefore reduces the strength and stiffness degradation as explained in chapter 1. The improvement on the cyclic response due to confinement can easily be observed comparing the hysteretic loops of specimen U6 (see Figure 2.12), with transverse reinforcement of Type B, with the hysteretic loops of specimen U2, with transverse reinforcement of Type A.

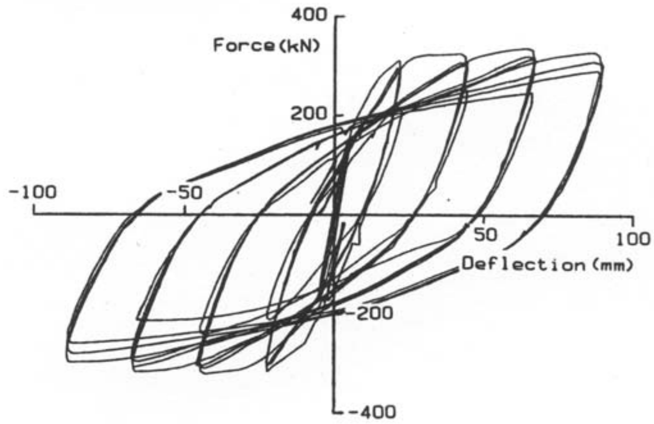


Figure 2.12 - Test results for specimen U6 (Saatcioglu and Ozcebe, 1989)

- Another negative effect of axial loading is the development of the well known second-order moments (*P-Δ effects* as it is generally known). It is obvious that as the level of axial loading rises, the more important are these effects and therefore, the larger are the strength requirements. The designer should avoid high levels of ductility demand in members subjected to high axial loading in order to minimize the risk of failure due to large second order moments. Underestimating the *P-Δ effects* is a frequent cause of failure as it leads to structural collapse due to lateral instability, particularly in buildings in which sidesway mechanisms are supposed to form.

2.1.2.2. Members with varying axial force

It is well known that overturning moments are present when a structure such as a two-dimensional frame is subjected to cyclic lateral loading. These give rise to axial forces in columns due to seismic loading to be compressive in one side of the frame and tensile in the opposite side. These forces increase from the interior to the exterior of the frame. For the columns in the interior of the frame this might not be critical, but for the external columns these forces cannot be neglected as they drastically may reduce the compressive forces or even inducing tensile forces when combined with the effect of the vertical component of the seismic motion.

Abrams (1987) conducted a test series on the influence of axial force variations on flexural behaviour of reinforced concrete columns. In the following the results in terms of moment-rotation relationship for two of the specimens tested are shown. Both specimens had the same geometry and reinforcement arrangement. Specimen C1 is a control specimen in which the axial load was kept constant at 310 kN (normalized axial load, ν=-0.1) whereas for specimen C4 the load varied linearly with the bending moment between 55 (ν = -0.02) to 588 kN (ν=-0.25).

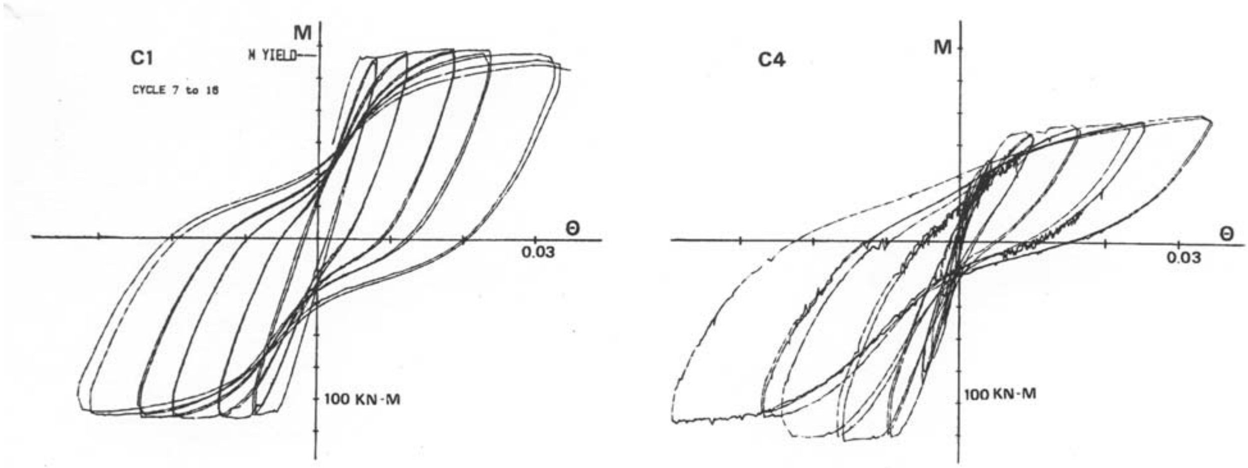


Figure 2.13 - Moment-rotation relationships for specimens C1 and C4 (Abrams, 1987)

Low compressive forces and/or tensile forces have a rather unfavourable effect on the cyclic behaviour of structural members, regarding the energy dissipation capacity and the stiffness development. This is due to the fact that these conditions impose restraints to the closure of the cracks due to flexure and shear (extension of *branch g)*; see Figure 2.4) in the course of cyclic loading. Therefore the pinching of the loops is more pronounced meaning loss of hysteretic energy dissipation capacity (See Figure 2.13, specimen C4). Also the concrete in the cracked cross-section is less effective for a longer time, which affects directly the stiffness of the member. It should also be noticed that further in the course of cyclic loading shear deformations become more important as explained in section 2.1.1. – Figure 2.6. Low compression forces or tensile forces reduce significantly the strength along the shear cracks and therefore increase the risk of failure due to sliding shear.

Moreover, from the axial load – bending moment interaction diagrams, it may be concluded that a significant loss of flexural strength takes place for low compression forces and/or tensile forces. This explains the reduction in the ordinates of the envelope curve of the upper half–cycle of the moment-rotation relationship for specimen C4.

The reverse of these tendencies was observed in the tests of Saatcioglu and Ozcebe (1989), in the specimens axially loaded. As appearing from Figure 2.11, the branches of the lower half-cycles are more inclined and achieve greater strength values (Specimens U2 and D2). The reasons for this behaviour were given when discussing the favourable effects of moderate axial compression in the previous paragraph. This explains the asymmetric pattern of the moment-rotation curve for specimen C4. It was concluded that the shape of the hysteretic loop is influenced by the range of axial force variation and also by the rate of change of axial force with lateral deflection (Abrams, 1987).

2.1.3. Members in biaxial flexure

During a seismic action the direction of the loading is permanently changing. Thus, very seldom, if ever, a structural member is submitted to a state of stress corresponding to bending in one of the principal directions. This applies especially to columns. This indicates the importance of the inelastic response of a member submitted to biaxial flexure. However, the interest in this subject is recent and the available experimental results are rather limited. This has to do with the complications coming from adding an extra parameter corresponding to the manner in which the histories of bending moments in two directions are combined. So the present-day knowledge of the inelastic behaviour of reinforced concrete members in biaxial flexure is very much behind the understanding of the behaviour under uniaxial cyclic flexure.

The primary effect of biaxial flexure is the strength degradation in one direction after an inelastic action in the orthogonal direction. Otani, S. Cheung, V.W.T. and Lai, S.S. (1980) carried out a test series, intending to investigate the effect of biaxial lateral load reversals on the cyclic response of reinforced concrete columns. They tested several specimens representing the part of the first-storeys columns between the foundation and the inflection point in the moment diagram. Specimen SP4 was first submitted to two uniaxial cycles slightly past yield (displacement ductility ratio, μ_y, of about 2.0). Afterwards the same specimen was loaded with eight uniaxial cycles at μ_y = 4.5. When cycling was repeated in the former direction at the same ductility ratio as before (μ_y = 2.0) a very noticeable degradation of strength in comparison to the last cycle was observed.

Saatcioglu and Ozcebe, 1989, also reached the same conclusion: Despite the fact that deformations prior to yielding do not noticeably affect the response in the orthogonal direction, inelastic cycles in one direction drastically reduce column strength in the other direction. In Figure 2.14 a significant drop of capacity in direction E-W may be observed comparing with the one for direction N-S, despite the member B1 was designed to have the same capacity in both directions. The authors estimated a drop of strength of about 20 to 30 % in column B1.

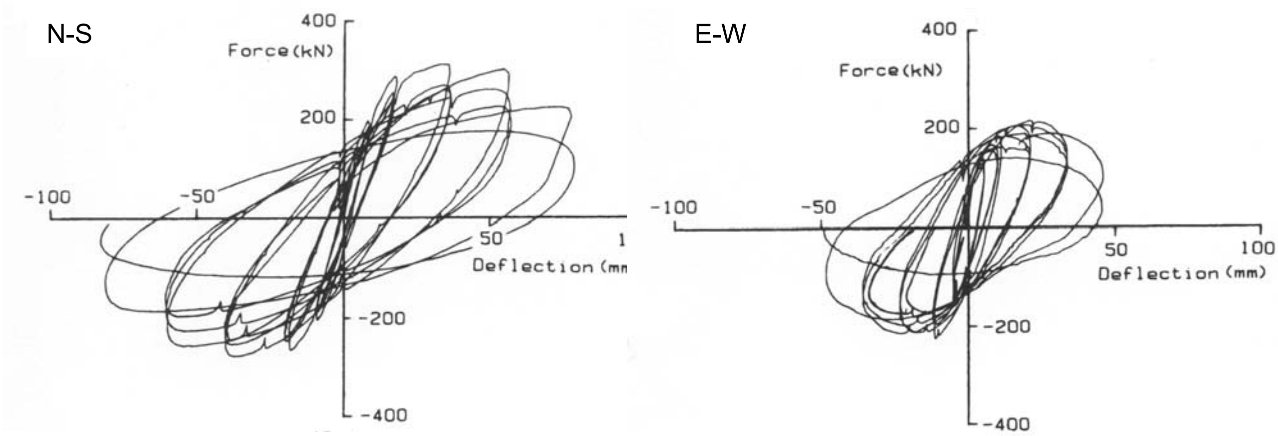


Figure 2.14 - Tests results of specimens B1 (Saatcioglu and Ozcebe, 1989)

The authors also showed that a member submitted to a deformation path in the diagonal direction significantly reduces the capacity in each of the principal axes even if the deformation path is meant to produce the same bending and shear in the two principal directions.

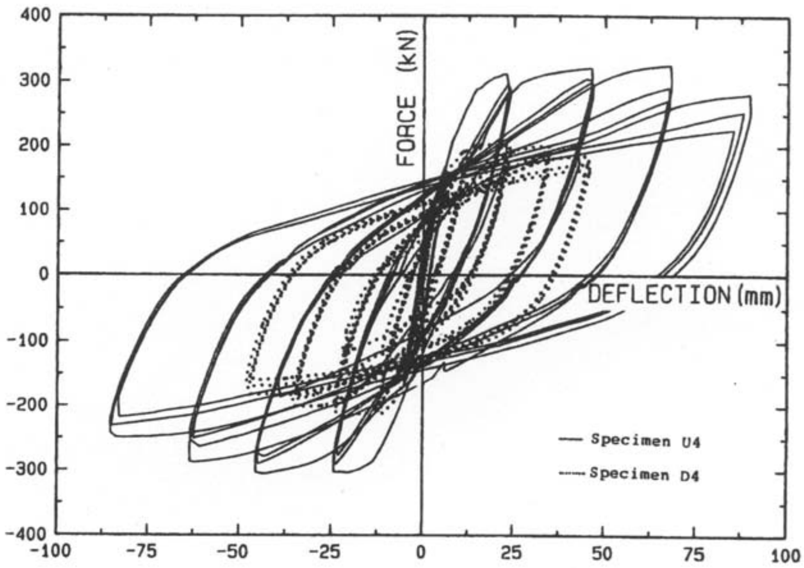


Figure 2.15 - Comparison of the responses of specimens U4 and D4 in the N-S direction (Saatcioglu and Ozcebe, 1989)

However when comparing the lateral load – top deflection hysteretic relationships in the direction of loading it is observed that the overall hysteretic characteristics are similar in terms of strength, stiffness and ductility (compare in Figure 2.11 the shape of the hysteretic curves for specimens U_i and specimens D_i). This feature was also observed by Umehara and Jirsa (1982). This led to the conclusion that the maximum capacities of the columns with diagonal unidirectional loading could be estimated by an interaction circle in case of a fully symmetric cross-section or by an ellipse, if the loading capacities are different in each principal direction. This inelastic diagram connects the maximum capacities of the columns under unidirectional loading along the principal axes.

In all experimental investigations mentioned above severe stiffness decay was observed after post-yield deformations in one of the two principal directions. This feature is well demonstrated in the hysteretic curve of specimen B1 tested by Saatcioglu and Ozcebe (Figure 2.14). A fast decay in stiffness leads to the unfavourable effect of larger lateral displacements, hence to a more pronounced second order effect (*P-Δ effect*).

The high rate of damage in biaxial flexure inducing significant drops in strength and stiffness after each cycle may be explained referring to the cracking mechanism. For a loading history of the type depicted in Figure 2.10 the direction of the load is permanently changing. Consequently parts of the cross-section will be in tension and others will be in compression for a significant period of time. This accelerates the degradation of stiffness and strength of the member due to the *Bauschinger effect* in the tensile steel and due to crushing of the concrete and consequent spalling of the concrete cover in the compression face. Also shear cracks will develop for both directions increasing the rate at which they intercept, meaning a faster degradation of the concrete core.

As it was mentioned in the beginning of this section, columns are structural elements exposed in a higher degree to biaxial cyclic flexure and its detrimental effects than for beams, particularly in frame structures. This means that biaxility of flexure imposes a more severe damage to columns than to beams. This highlights even more the correct assessment of the effect of biaxial flexure so that the capacity design procedures (see chapter 3) can be accurately applied in structural design.

2.2. Members with shear-dominated behaviour

The discussion so far has referred to flexure-dominated members, i.e. to slender members. If the slenderness of the element drops to a certain level, the ultimate load is governed by shear forces. It is known that this type of behaviour is characterized by very low ductility and, in general, by poor performance under cyclic loading. This has been confirmed in the field after the spectacular shear failures of short columns observed after the 1968 Tokachi-Oki and the 1972 Managua earthquakes, which showed a rather brittle behaviour of these type of members.

The slenderness parameter l/h, in which l is the length of the member and h its depth, is often used to trace the border between these to types of structural members regarding their structural behaviour. The reference value for slenderness that separates the two types of behaviour is around 4. In frame structures low slenderness elements are either deep beams (high h) or short columns (low l). As already mentioned, the tendency nowadays in seismic design is to limit the strength of beams (weak beam / strong column criterion). Therefore deep beams are very uncommon. On the other hand short columns appear in frames frequently. This is done intentionally in the design of the frame or unintentionally as in the case of slender columns the effective length of which is reduced by infill masonry walls up to a certain height in the frame. The latter case has dangerous consequences since the behaviour of shear–dominated members is substantially different from the one described in the previous section, as will be shown next.

Authors often designate members with shear-dominated behaviour as members of *low shear span ratio*. The shear span ratio α, is defined as $\alpha = M/Vh$. It is obvious that for an antisymmetrically loaded short-column ($V = 2 \cdot M/l$), the shear span ratio is less than 2.

Garstka, B, W.B. Krätzig and F. Stangeberg (1993) showed that as the shear span ratio decreases below the critical value of about 2.0, the monotonic load-deformation curve gradually shifts from the ductile mode of the flexure-dominated behaviour to the brittle mode of the shear-dominated behaviour. In fact, it may be seen in Figure

2.16 that for members with low shear span ratio (in the figure *a* stands for the length of the member) the load-deformation curve exhibits a long nearly linear initial branch that softens smoothly to a well-defined ultimate peak strength, which is followed by a rather steep descending branch. However the differences for those three specimens were only in the shape of the deformation response since all the three specimens failed at the same value of the end moment. Then, it is evident, that the lower the shear span ratio of the member the closer the shape of the monotonic load-deformation curve resembles that of concrete in compression. This has to do with the effect of the compressed diagonal strut, which carries the shear force. The damage imposed on the member, i.e. crushing after the occurrence of yielding of the longitudinal bars and gradual reduction in the depth of the compression zone, still takes place at the end section, but not normal to it, as in slender members, but with an inclination perpendicular to the compressed diagonal. As the shear span ratio drops well below the limiting value of 2.0 the behaviour is more and more controlled by the concrete along the compressed diagonal (CEB 1994).

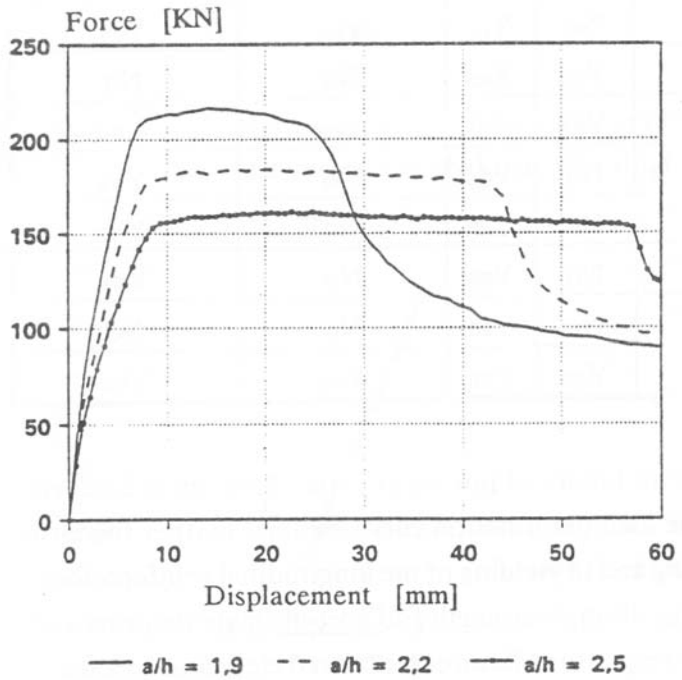


Figure 2.16 - Monotonic force-deflection curves at different shear span ratios
(Garstka, B, W.B. Krätzig and F. Stangeberg,1993)

Tests by K. Maruyama, H. Ramirez and J.O. Jirsa (1984)

These authors undertook a test series with the purpose of investigating the behaviour of short-columns subjected to different cyclic lateral loading histories. The geometry of the specimens was not changed (see Figure 2.17).

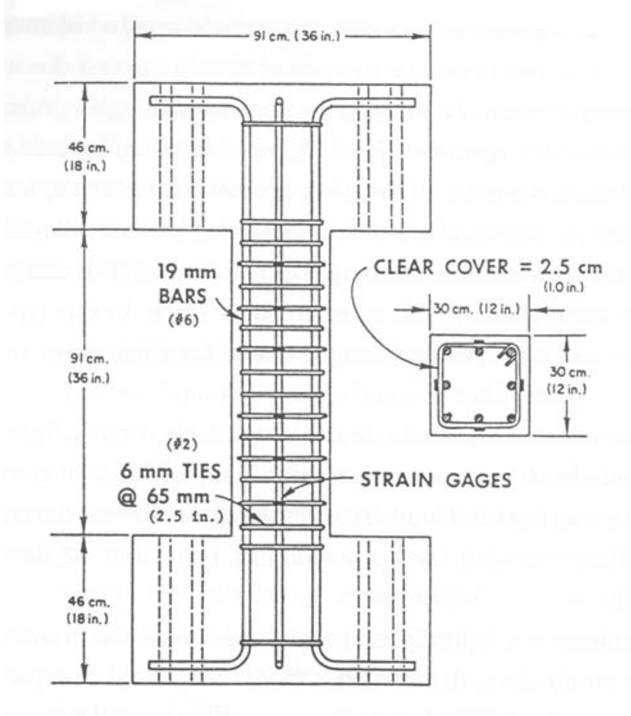


Figure 2.17 - Geometry of the specimens tested
(K. Maruyama, H. Ramirez and J.O. Jirsa,1984)

In Figure 2.18 the test result for specimen O-U is shown in terms of force-deflection relationship of a column without axial load submitted to unidirectional loading. The load history consisted of 3 cycles in which the peak displacement was Δ_y followed by another 3 cycles in which the peak displacement was $2\Delta_y$ and so on until the last 3 cycles had a peak displacement of $4\Delta_y$.

It is evident that the hysteresis loops are narrower than in the flexure-dominated slender members and attain a pronounced inverted *S*-shape, i.e. pronounced pinching. This indicates a very poor hysteretic energy dissipation capacity.

The shape of the force-deflection relationships for members which behaviour is shear-dominated has to do with the role of the stirrups (CEB, 1994). Until the first inclined cracks form, the stirrups do not carry any shear. Therefore unloading and reloading prior to the opening of the cracks is almost elastic. After the yield deflection, the shear transferred across the cracks consists of contributions of the compressed concrete between the cracks, stirrups crossing the inclined cracks and aggregate interlock forces on the surface of the cracks (Wight and Sozen, 1973). Dowel forces may have a small effect. Also the bond deterioration between stirrups

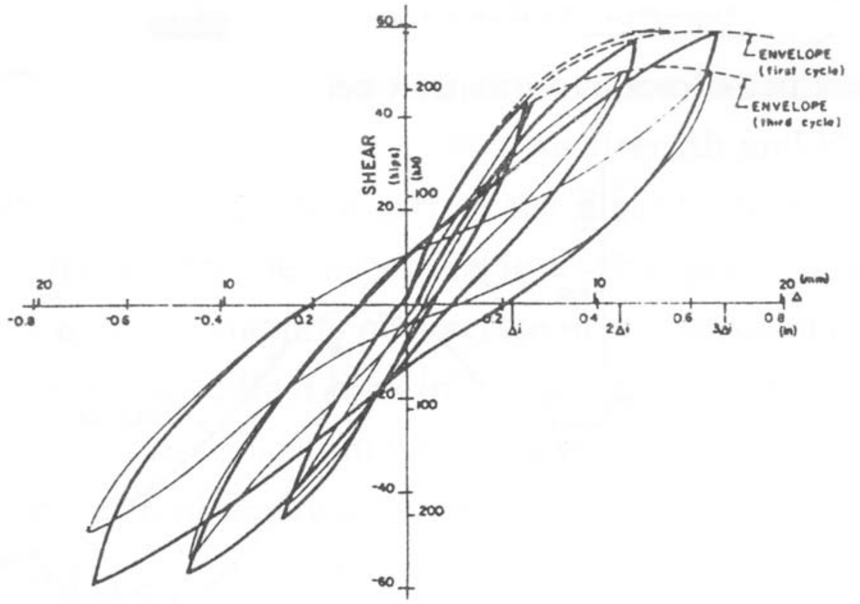


Figure 2.18 - Force-deflection response in test O-U without axial load
(K. Maruyama, H. Ramirez and J.O. Jirsa, 1984)

and the surrounding concrete contributes to larger tensile forces on the stirrups. After yielding, tensile strains tend to accumulate with cycling which means that the inclined cracks remain open for a longer time. This increase in strain means that the inclined cracks open wider in each successive cycle and, as the width of the inclined cracks increases, the pinching effect becomes more and more pronounced leading to a corresponding decrease in shear strength and stiffness and a reduction in the energy dissipation capacity of the member.

Loops are narrow because the behaviour is controlled by the concrete along the compressed diagonal, which leads to a rather limited capacity of deformation compared to flexure-dominated members in which the longitudinal reinforcement has a larger contribution to the overall deformation.

The work carried out by Woodward and Jirsa, 1984, at the University of Texas, is useful for the understanding of the effect of both transverse and longitudinal reinforcement in the behaviour of shear-dominated members. These authors concluded that increasing the ratio of transverse reinforcement increases the energy dissipation capacity and the deformation capacity at ultimate strength as for slender members. However the parameter ultimate strength is left unaffected or improves slightly. This indicates that the transverse reinforcement has a rather indirect role in the cyclic behaviour of short elements. Most part of the ultimate strength is developed before the formation of the inclined cracks. After cracking, the shear resistance of the member is strongly related to the effectiveness of the aggregate interlock along the inclined cracks (Figure 2.19). The primary function of the stirrups is to control the widths of the inclined cracks to maintain the effectiveness of the aggregate interlock (Woodward and Jirsa, 1984). Thus, the monotonic force-deformation curve is an upper-bound envelope of the cyclic response.

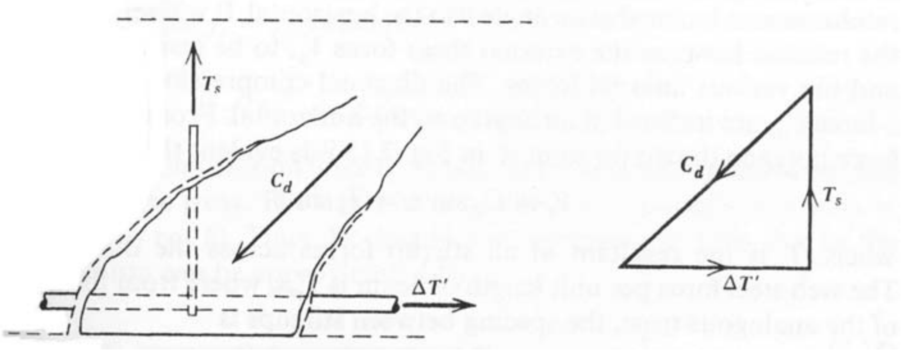


Figure 2.19 - The mechanism of shear resistance (Park and Paulay, 1975)

It was also concluded in this experimental investigation that, for the same reasons as in slender members, an increase of the longitudinal reinforcement leads to larger stiffness and strength degradation and also reduces the energy dissipation capacity.

An important observation from the experimental investigation carried out by K. Maruyama, H. Ramirez and J.O. Jirsa, 1984 was the spreading of the damage throughout the whole length of the member instead of being concentrated in the regions with high bending moments, as in slender members. With no axial load, severe diagonal shear cracks formed at both ends of the column. With added cycles or increase in the deflection magnitude, cracks extended and new cracks appeared. At failure the entire column was covered with cracks and several large cracks dominated the pattern (K. Maruyama, H. Ramirez and J.O. Jirsa, 1984).

The above-mentioned observation leads to the conclusion that failure may happen in any region of a low shear span ratio member. The failure of this type of elements normally starts with splitting of the concrete along the compressed diagonals and crushing of the outermost concrete fibres. Interception of the cracks in the course of cycling contributes to the stiffness and strength deterioration. Typically failure is associated with the collapse of the member due to the excess of the load-carrying capacity of one of the compressed diagonals, which takes place in a rather brittle and explosive way.

Research has been carried out in recent years with the intention of improving seismic performance of low span ratio members, by finding new arrangements of reinforcement. The greater effectiveness of inclined shear reinforcement compared to the vertical one is well known since, the former is in the principal directions of the diagonal stress field of the shear-dominated member. Bearing this in mind, Park and Paulay, 1975 have proposed the use of cross-inclined diagonal bars (Figure 2.20 b)) and Tegos & Penelis, 1988, suggested the use of multiple cross-inclined bars, forming a rhombic truss (Figure 2.20 c)). In Figure 2.20 the arrangements of reinforcement and the corresponding crack pattern at failure are depicted. Test results have indicated an improvement on the shear capacity, as well as in stiffness and energy dissipation (Penelis and Kappos, 1997). It seems that even better behaviour would be achieved by means of closely spaced diagonal reinforcement.

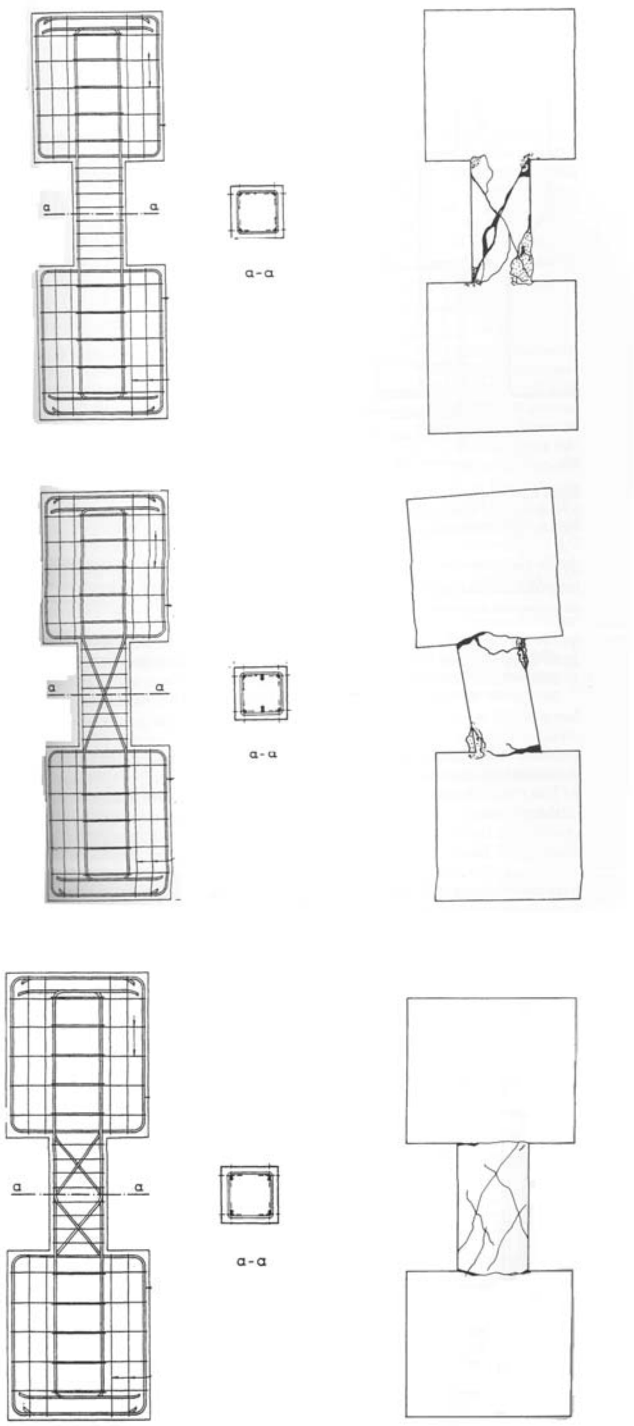
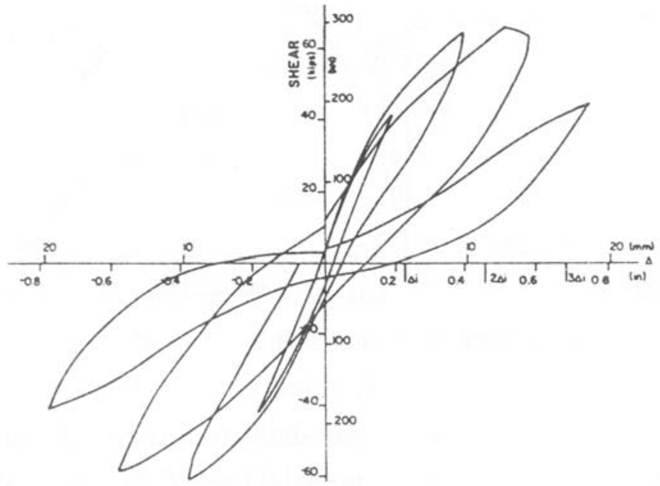


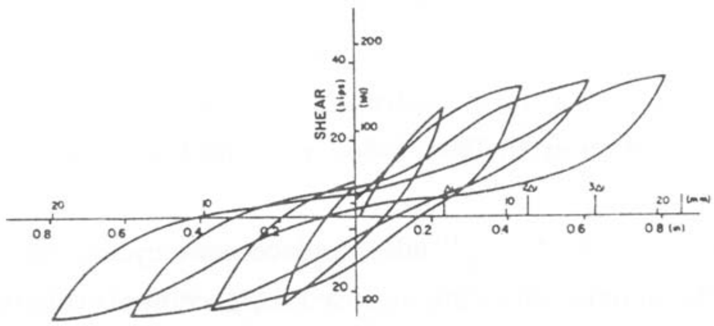
Figure 2.20 - a) Conventional reinforcement (closely spaced ties); b) Bidiagonal reinforcement and c) Rhombic reinforcement (Penelis and Kappos, 1997)

2.2.1. The effect of axial forces

The tests carried out by K. Maruyama, H. Ramirez and J.O. Jirsa, 1984, also clarify the effect of axial force. The specimens in Figure 2.21 were submitted to the same loading history as the specimen in Figure 2.18, but with different levels of axial force applied.



a)



b)

Figure 2.21 - Effect of axial load on cyclic response a) ν = -0.19 (compression) and b) ν = 0.12 (tension) (K. Maruyama, H. Ramirez and J.O. Jirsa, 1984)

It may be concluded, comparing Figure 2.18 with 2.21 a), that the presence of low-to-medium compressive forces increases the ultimate strength of a low shear span ratio member. This is due to the increase in the depth of the compression zone and so, in this zone, the shear may be transferred by inclined compression. However this seems to be the only favourable effect since the rate of stiffness and shear strength decay is much larger in the post-yield deflections for the compressed member. Furthermore the capacity of energy dissipation is severely affected as seen in Figure 2.21, noting that the axially compressed specimen has a larger *pinching effect*. Explanation for this lies in the fact that added compression on the compressed diagonal strut accelerates even more the splitting along the diagonal cracks and the spalling of the concrete cover. Moreover the stirrups are already in tension due to lateral expansion from the compressive axial load. Thus, the stirrups are not as effective in providing confinement to the core at high levels of deformation, and in maintaining the shear capacity as in members without axial load. As a consequence the rate of degradation of the inner structure of the member is higher for compressed members.

Axial tension has the reversed effects of axial compression for low shear span ratio members under cyclic loading. In fact, it may be concluded comparing Figures 2.18 and 2.21 b), that the shear force required to attain a given deformation decreases for members in tension. This may be explained by the fact that axial tension diminishes the contribution of the effect of the aggregate interlock in the shear resistance mechanism after the inclined cracks have formed. This also explains the reduced stiffness of the response even for the first cycle. However less degradation of stiffness, strength and energy dissipation capacity are observed. In fact the shape of the hysteresis loops resembles the ones of the flexure-dominated members. This is because the forces in the stirrups are not mobilized until large lateral deformations close the horizontal cracks from tension, and diagonal cracks form. Therefore, the shape of the hysteresis loops is more stable enabling the member to reach higher levels of ductility but at lower levels of strength and stiffness.

2.2.2. Members in biaxial shear

The tests performed at the University of Texas also included a series of biaxial tests to investigate the effect on biaxial shear in low span ratio reinforced concrete members. The tests to be reported in the following were of two types according to the direction of the load:

- Alternate cyclic loading in both directions (Test O-B4). This test was conducted to examine the influence of previous cyclic loading in an orthogonal direction and
- Cyclic loading in one direction with permanent deflection in the orthogonal direction (Tests O-U2 and O-U4)

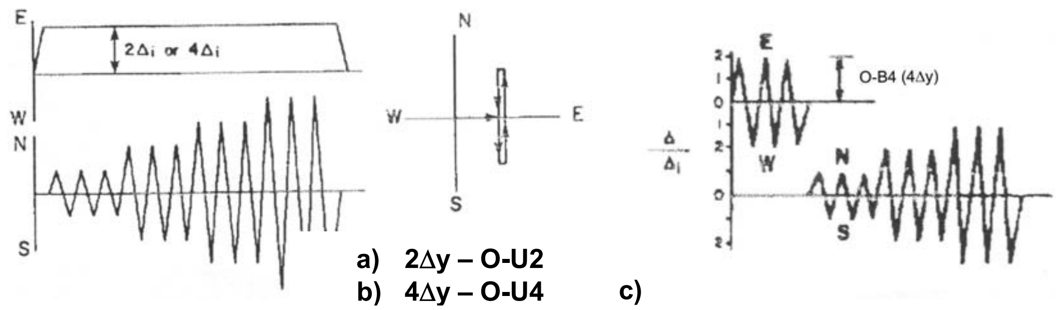


Figure 2.22 - Loading histories a) O-U2 b) O-U4 and c) O-B4
(K. Maruyama, H. Ramirez and J.O. Jirsa, 1984)

Figure 2.23 exhibits the response for the three specimens in terms of the hysteretic relationships.

As already mentioned the geometry of the specimens tested was kept unchanged. However according to the loading histories the cyclic response was different. This fact is evident when comparing the responses referring to the N-S direction for the three specimens and the response curve of the control specimen O-U (Figure 2.18).

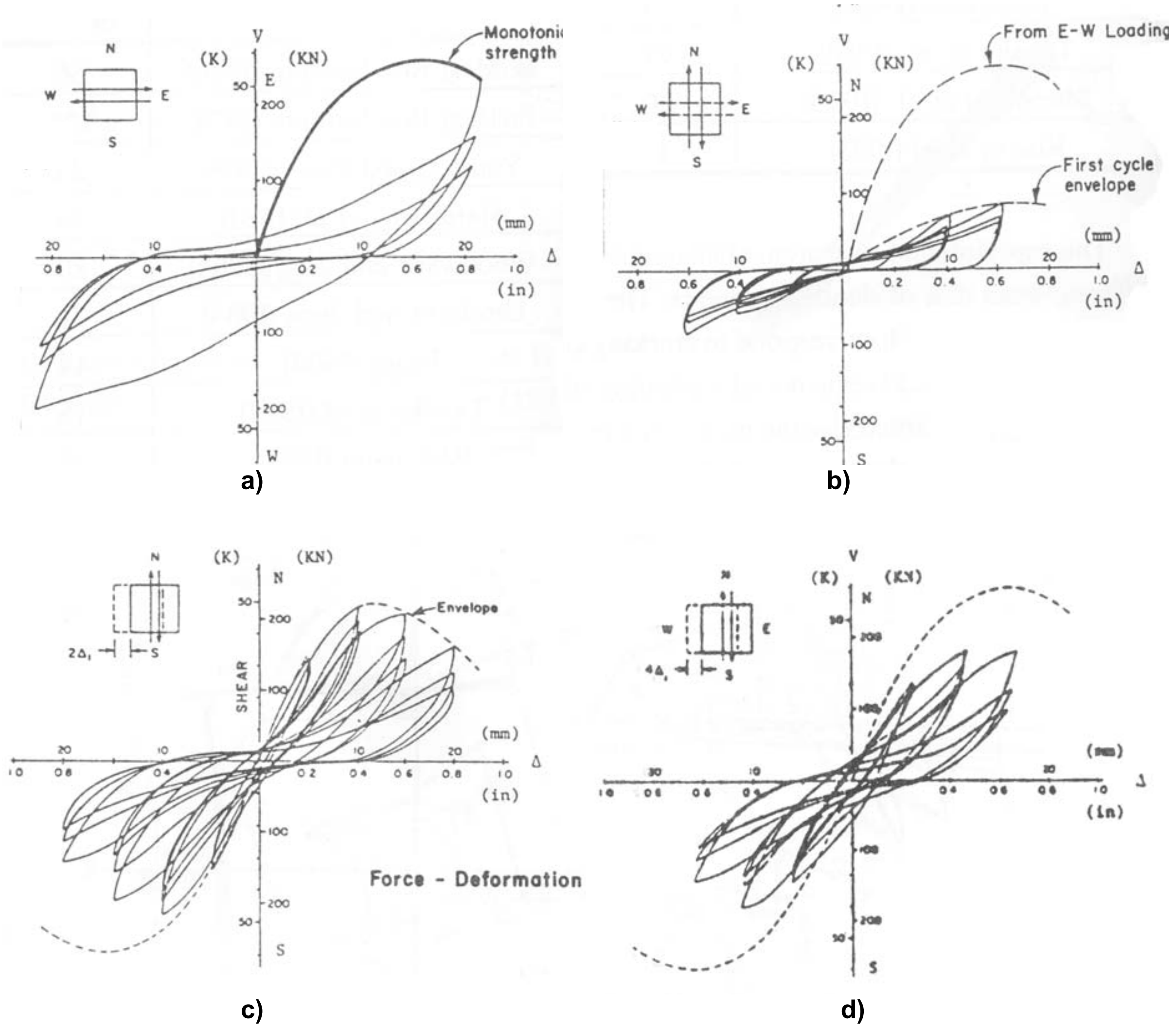


Figure 2.23 - Force-deflection response a) O-B4 (E-W direction) b) O-B4 (N-S direction) c) O-U2 and d) O-U4 (K. Maruyama, H. Ramirez and J.O. Jirsa, 1984)

- It appears that the N-S strength of specimen O-B4 was only a fraction of the strength under unidirectional loading as shown in Figure 2.18. The only difference in these two specimens was that Specimen O-B4 had already been submitted to cyclic loading in the orthogonal direction with a magnitude of $4\Delta_y$ since the loading in the N-S direction was developed in the same way as for the control specimen O-U. Moreover the response of Specimen O-B4 in the N--S direction showed lower strength than the one of specimen O-U4 and with a more pronounced *pinching effect*. This leads to the same conclusion as for the biaxial flexure case (section 2.1.3): cycling in one direction with a magnitude larger than the yield displacement, Δ_y, severely affects the strength and energy dissipation capacity in the orthogonal direction. The cycling nature of the load is the governing factor for the strength degradation in the orthogonal direction. This may be concluded referring to the fact that specimen O-U4, with a superior response, had also been exposed to a deflection in the orthogonal direction of $4\Delta_y$, but of permanent nature instead of cycling. This has to do with the fact that cycling not only contributes to increase the damage already induced in the members on the previous cycle by the successive opening of the cracks, but also spreads further damage as new cracks form.

- The strength in the N-S direction for specimen O-U2 is larger than that of specimen O-U4. The only difference between these two tests lies in the magnitude of the permanent deflection in the E-W direction. The specimen with larger permanent deflection exhibited larger strength decay in the N-S direction. Thus, it can be concluded that the larger the inelastic action in one direction the larger the strength decay in the orthogonal direction.

Umehara and Jirsa (1984) also concluded that previous loading in perpendicular directions does not significantly affect the maximum shear strength of the short shear span ratio members unless the maximum deflection in the previous loading exceeds the deflection at which the maximum shear strength of the members under unidirectional loading is reached. The same authors reached the same conclusion as in 1982 (already referred to in the discussion regarding the biaxial state of flexure, section 2.1.3): as in slender members, the maximum capacity for low shear span ratio members with diagonal unidirectional loading may be estimated using an interaction circle or ellipse (for unsymmetric cross-sections) connecting the maximum capacities of the columns under unidirectional loading along the principal axes.

2.3. Joints

The term *joint* refers to the regions where structural elements (columns and beams) intercept. It is now recognized that joints might be critical regions in reinforced concrete frames submitted to cyclic loading. However until the late 1970's the seismic provisions in all countries were based on the erroneous assumption that conditions within the joint, which often have somewhat larger dimensions than the members it joins, were not critical (Park and Paulay, 1975). This assumption was supported by the observations in field after strong earthquakes that showed little evidence of the contribution of joint failures for the major damage or collapse of structures. Paulay and Priestley (1992) explained these observations as being due to the inferior standard of design of beams and particularly poor detailing of columns rather than attributing *apriori* a non-critical performance to the joints.

Only in recent years the behaviour of joints submitted to seismic action has been a subject of interest. There is still a substantial debate about the joint resistance mechanisms under cyclic loading as the present knowledge is much behind the one referring to members. Moreover, experimental results about the cyclic behaviour of reinforced concrete joints are very scarce, since detailed experimental investigations are very recent. This is reflected in the significant differences in both the design approach and reinforcement detailing in modern codes.

Thus, in the following a qualitative description of the cyclic behaviour of concrete will be given with the intention of providing information about its main features and the parameters influencing it. Bonacci, Filippou and Pantazopoulou, CEB (1994), provided a critical compilation of available experimental results and design recommendations from several countries with the purpose of establishing the current state of the art. The interested reader is suggested to consult this reference for further study of the subject.

2.3.1. Qualitative description of the mechanics of joints

The behaviour of a joint is characterized by a complex interaction of shear, bond and confinement mechanisms taking place in a quite limited area. Still significant differences exist among seismic codes with regard to the shear transfer mechanisms assumed. In the following, the simple approach suggested by Paulay and Priestley, 1992 and adopted by the Standards Association of New Zealand is going to be presented.

Considering the overall statics of a given two-dimensional frame as shown in Figure 2.24, it appears that lateral loading imposes such a bending moment field in the beams and columns that moments with the same magnitude but of opposite sign will take place on parallel faces of the joint. As a consequence, the joint region is subjected to horizontal and vertical shear forces whose magnitude is l_c/d_b times the maximum shear force in the columns and l_b/d_c times the maximum shear force in the beams, respectively (see the meaning of the symbols in Figure 2.24).

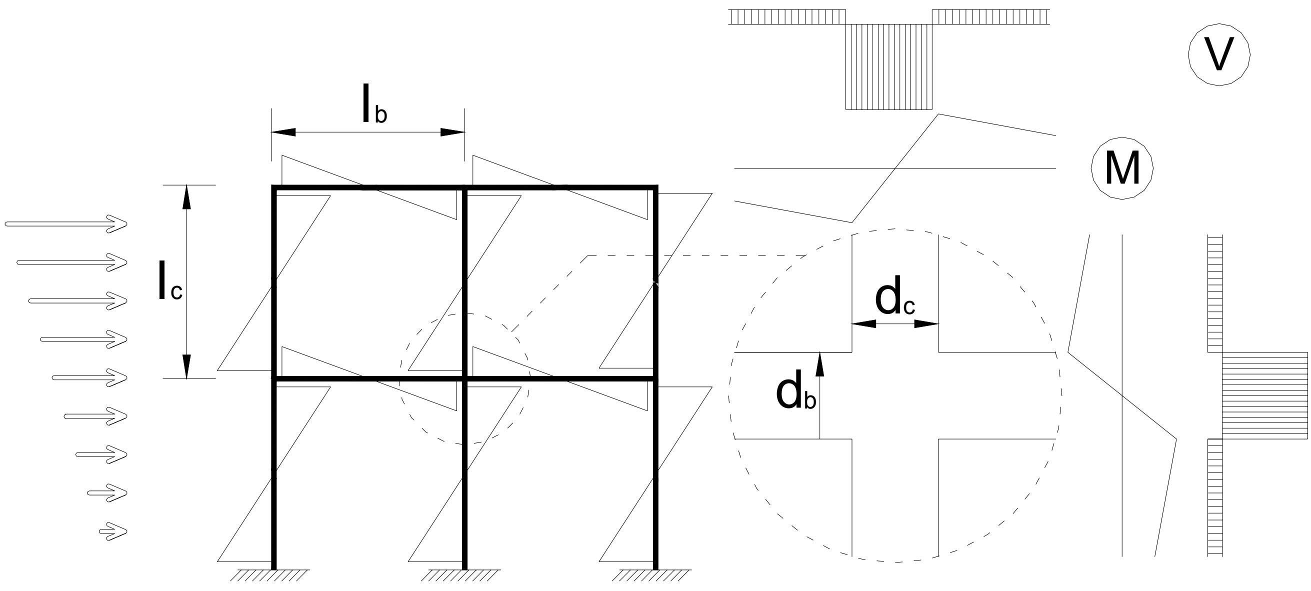


Figure 2.24 - Statics of laterally loaded frame;
Detail: Moments and shear gradient through an interior joint

Consider now the equilibrium of the interior of the joint, represented in Figure 2.25 a). It may be seen that the joint core is submitted to two types of actions that combined are generally known as the *joint shear*:

- Concrete flexural compression from beams and columns at the opposite corner of the joint (Figure 2.25 b)) and
- Shear flow along its perimeter from beam and column bars by means of bond forces (Figure 2.25 c))

The resistance mechanism is composed by a compressed diagonal of concrete roughly limited by the neutral axes of the end sections of the members (Figure 2.25 d)) and by diagonal compression field– truss mechanism – consisting of horizontal hoops, intermediate column bars (Figure 2.25 f)) and inclined compressed concrete between shear cracks (Figure 2.25 e)).

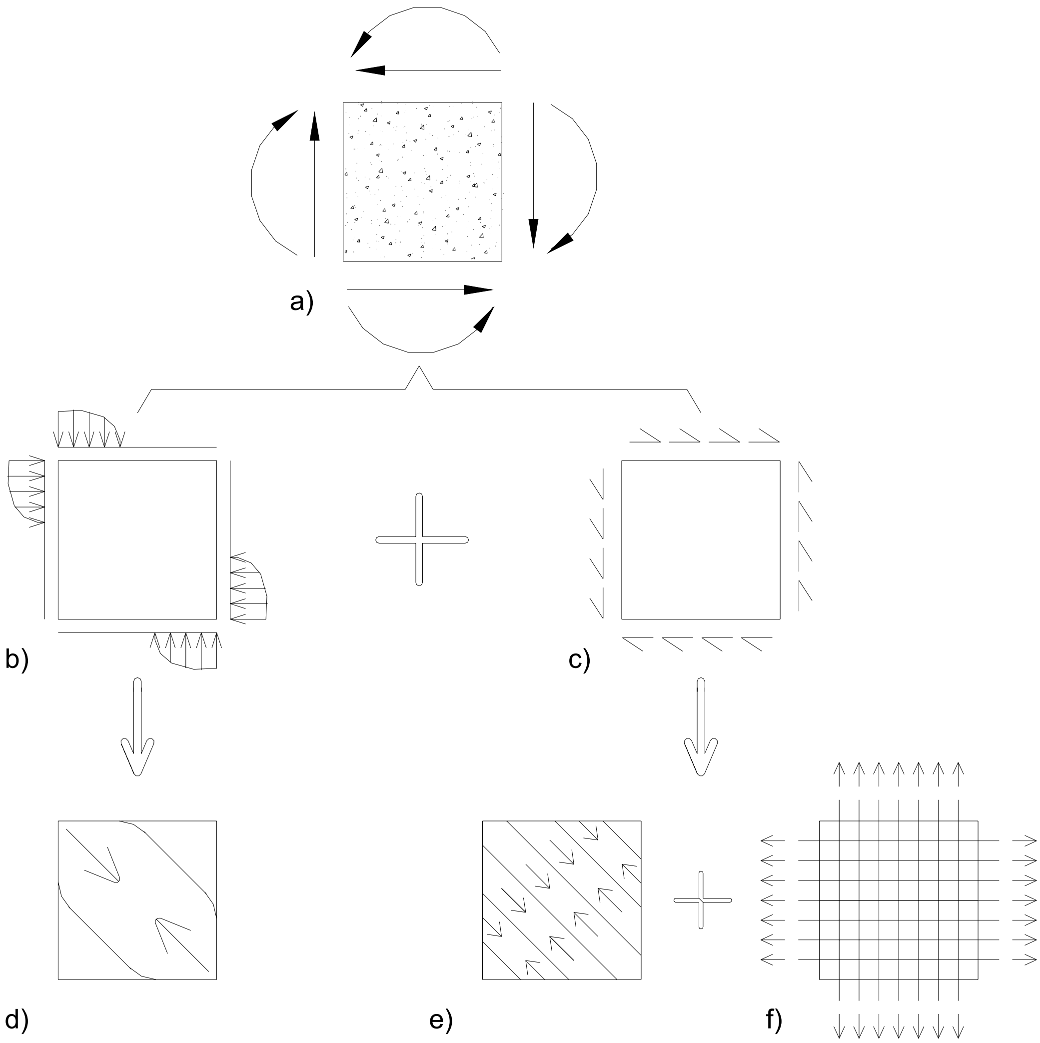


Figure 2.25 - Actions on a interior joint and the corresponding resistance mechanism according to Paulay and Priestley, 1992

The main component of the resistance mechanism is the compressed diagonal strut, which carries a substantial portion of the joint shear. The rest of the joint shear is transmitted to the joint core through the bond between the longitudinal reinforcement of beams/columns and the surrounding concrete and, therefore, absorbed by the truss mechanism. Depending on the magnitude of the bond forces, diagonal tension cracking takes place. The main crack is developed along the compressed strut but other cracks parallel to it form as well. In Figure 2.26 a crack pattern typical of joint shear is clearly seen.

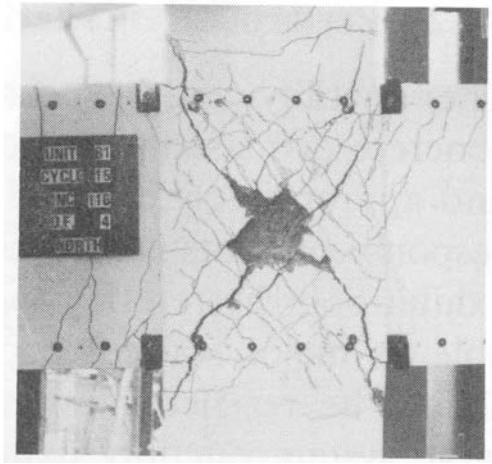


Figure 2.26 - Crack pattern of a joint (Paulay and Priestley, 1992)

To prevent shear failure by diagonal tension, both horizontal and vertical reinforcement are required. Such reinforcement enables a diagonal compression field to be mobilized as shown in Figure 2.25 e). This leads to the conclusion that the amount of reinforcement may be significantly higher than would normally be provided by the extension of the reinforcement of beams and columns into the joint core. This is particularly true in the case of joints whose columns are low axially loaded.

2.3.2. Influence of cyclic loading

As demonstrated in the previous section, the joint resistance mechanism depends on bond forces along its perimeter so that a truss mechanism can be mobilized and on a compressed diagonal strut between corners. These are rather brittle modes of behaviour, which explains the very limited capacity that joints have in dissipating energy and maintaining their strength.

The contribution of the diagonal compressed strut is significant during the first cycle in the inelastic range. However it deteriorates with the increase of the inelastic loading cycles. This is due to the fact that cycling at high levels of inelastic deformation causes permanent elongation on the beam bars and leads to full depth open cracks at the beam-joint interface. This was already discussed in section 2.1. Under these conditions flexural compression from the beams become negligible. The compressive forces are then transmitted to the longitudinal bars of the beams, which significantly increase the bond stresses along the horizontal perimeters of the joint core. This leads to a drastic reduction in the contribution of the concrete strut to the transfer of horizontal joint shear and a consequent increase in the contribution of the truss mechanism. The mobilization of the truss mechanism depends intimately on the effectiveness of bond between the steel bars and the surrounding concrete. As it was already discussed in section 1.3, bond has a very poor response in terms of energy dissipation, stiffness and strength degradation under inelastic cycling. Thus, it can be concluded that the development of plastic hinges in the end sections of the beams seriously affects the ability of the joint to resist in a stable manner the induced shear forces. Again, joints whose columns are low axially loaded are the most sensitive to bond deterioration since compression helps to maintain the bond mechanism.

The foregoing serves to emphasize the need to take special precautions to prevent premature bond deterioration in joints under seismic loads. Adequate confinement of the joint core significantly improves the bond performance under seismic conditions (Paulay and Priestley, 1992). Confinement may be provided by axial compression of the column and/or by means of reinforcement using the intermediate column bars as these members are supposed to remain in the elastic domain. Moreover, confinement improves the performance of the compressed diagonal strut.

Yielding of the longitudinal bars of the beam leads to another form of degradation of the shear resistance of the joint: As the horizontal bars yield in tension, the shear cracks due to diagonal tension tend to remain open being locked on the extended steel bars. This contributes to a rapid degradation of the shear resistance in the truss mechanism with cycling due to the successive drop in the friction forces along the shear cracks. Once again this effect may be diminished taking advantage of the intermediate column bars which are supposed to remain in the elastic range and therefore may contribute throughout the whole seismic action to the closing of the shear cracks.

The reversals in the loading also contribute to the spreading of the cracks in orthogonal directions. As seen in the case of members, this leads to successive degradation in the strength of the compressive diagonal struts since the closing of the cracks is not completely effective because the surfaces may not come into full contact. The damage induced by cross-inclined cracking also affects adversely the bond conditions of the longitudinal bars intercepting the concrete core. For the control of this effect, confinement plays an important role as well as it keeps the inner structure of the concrete member preserved by controlling sliding along the cracks.

Failure of the joint is due to the inability of any of the “sub-mechanisms” depicted in Figure 2.25 d), e) and f) to carry successfully the load they are meant to sustain. It follows then that three different sources for joint failure can be pointed out:

- Failure of the compressed diagonal strut;
- Failure due to loss of bond resistance along the joint boundary and

- Failure due to inability to develop a truss mechanism that can carry the diagonal tension by the premature yielding of the longitudinal bars intercepting the core (Figure 2.27).

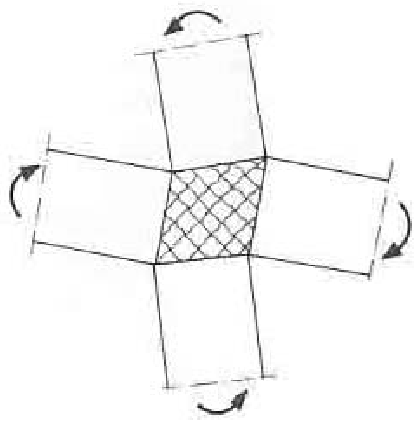


Figure 2.27 - Shear failure due to premature yielding of the joint reinforcement (Penelis and Kappos, 1997)

One of the means to control the first and the second mode of failure is the obvious impact of increasing the joint dimensions. As explained before, the design philosophy nowadays leads to the weak beam/strong column concept. Therefore it is only reasonable to increase the joint dimensions by means of a greater depth of the columns. This has the dual effect of leading to less shear stresses in the joint core and also to lower bond demand along the beam bars passing through the joint.

Additionally to adequate confinement, good anchorage of the longitudinal beam bars is a decisive factor for the maintenance of the bond strength throughout the seismic loading. As known, this is achieved by an appropriate anchorage length and configuration and by limiting the diameter of the bar.

3. Conclusions / Implications on Seismic Design

As mentioned before, the *design philosophy* nowadays in the seismic design is to provide the structure with properties that ensure the dissipation of the energy induced by an earthquake. The more energy dissipated, the less strength required by the structure. This means not only safer structures but also more economic ones.

Regions of the primary lateral force resisting mechanism are carefully selected, designed and detailed so that they can dissipate as much as possible the energy transmitted to the structure by the base motions. In frames these regions are generally known as *plastic hinges* and together they form the *energy dissipation mechanism of the structure*. The energy is dissipated taking advantage of the ductile properties of the plastic hinges, i.e. their ability to maintain strength in the inelastic range and absorb energy by hysteretic behaviour.

The successful performance of the structure in sustaining large imposed base motions depends mainly on the ability of the energy dissipation mechanism of the structure to hold during the entire seismic action. This is achieved assuring that:

- Each plastic hinge is designed to have strength as close as possible to the required strength and is carefully detailed to maintain its ductility. The former requirement is due to the fact that the lower the strength the larger the ductility might be (See Figure 1 b) of the Preface);
- The only mode of failure of a member containing a plastic hinge is the one corresponding to the development of the capacity of the plastic hinge. Therefore all the other modes of failure are inhibited by providing them with strength greater than the capacity of the plastic hinge;
- In the same way, regions not suited to dissipate energy in a stable manner are protected by ensuring that their strengths exceed the requirements from the development of the plastic hinge strength. Therefore these regions are designed to remain elastic.

These three requirements are the basis for the so-called *capacity design procedure* and their applicability is exemplified in the following to the simple case of a multi-storey two--dimensional frame. The plastic hinges of the mechanisms to be considered for this frame are supposed to dissipate energy by means of inelastic rotations in the end-section of members.

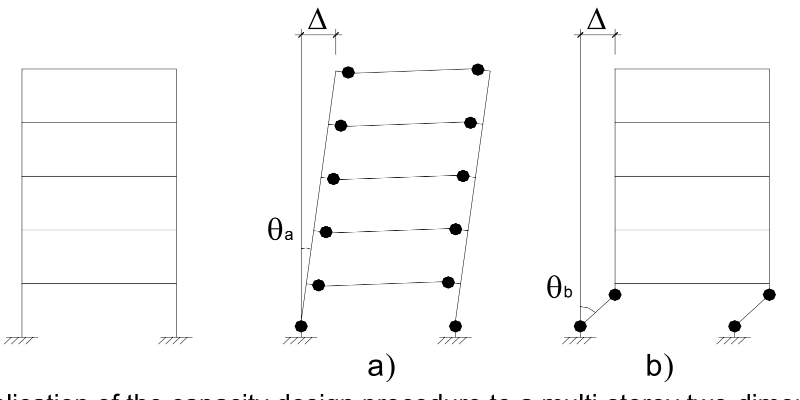


Figure 3.1 - Application of the capacity design procedure to a multi-storey two-dimensional frame (adapted form Paulay and Priestley, 1992)

The major steps are summarized in the following (Paulay and Priestley, 1992):

i. A kinematical admissible plastic mechanism at failure is chosen so that the maximum energy may be dissipated. Two fundamental criteria are used to define the most effective mechanism: Firstly the overall displacement of the structure should be achieved with the smallest inelastic deformation of the plastic hinges. Therefore the mechanism in Figure 3.1 a) is preferable since the inelastic rotations of the plastic hinges are considerably less than in the mechanism of Figure 3.1 b) for the same overall displacement Δ. Secondly in order to dissipate as much of the energy transmitted by the earthquake as possible a significant number of plastic hinges should form before collapse. For the present case it is obvious that the mechanism in Figure 3.1 a) is more advantageous.

ii. Parts of the structure intended to remain elastic are designed with respect to the situation of feasible action causing the development of the strength of the plastic hinges. Considering the frame in Figure 3.1, this means that the strength of the regions that are not plastic hinges must well exceed the required strength corresponding to the onset of the plastic moments in the plastic hinges. To assure this, a factor larger than unity is used, the *overstrength factor*. The latter is to take into account the variability of the yield stress on the reinforcement and the probability of strain-hardening effects that increase the strength of the plastic hinge after yielding.

iii. The nature and quality of detailing must be clearly distinct between the regions assigned to be plastic hinges and those which are to remain in the elastic domain.

It was seen throughout Chapters 1 and 2 that, as long as provided with the adequate reinforcement, flexural yielding mechanisms are the ones presenting a more reliable ductile behaviour opposing to the shear and bond-slip mechanisms. Therefore the plastic hinges in a capacity-designed structure should dissipate energy by means of flexural yielding rather than exploring inelastic shear and bond-slip deformations. This means that the designer should always assure that failure at the plastic hinge takes place by flexure before the exhaustion of shear capacity as well as bond strength.

Capacity design is an important design tool which allows the engineer to choose and implement a satisfactory response despite the characteristics of the earthquake to which the structure is going to be submitted.

As a final remark in this section, it should be noted that in this approach *damage* is accepted *a priori* in a quite realistic manner. Damage is going to be located in the areas where plastic hinges are expected, and will be limited by the ductility demands imposed by the designer in these regions. Therefore this design procedure not only enables the location of the damage in zones easy to repair but also the prediction of the level of damage to be imposed on the structure.

3.1. Strong column – weak beam concept

The strong column–weak beam concept is the corollary of the capacity design procedure and it is of fundamental importance in the design of structures whose seismic resistance system is composed by ductile frames. Considering the structural functions and modes of behaviour of beams and columns, this concept establishes that the energy dissipation mechanism of the structure is composed by flexural plastic hinges taking place in beams and avoided in columns. Therefore, from the discussion in the last section, it is obvious that the strength of the beams is limited to the plastic hinge capacity and the columns are supposed to remain in the elastic domain. Column design moments are, according to this concept, derived at beam-column joints with respect to the actual resisting moments of the plastic hinges in the beams.

Columns are traditionally designed to withstand axial loads by compression from the weight of the structure and from “live loads”, gravity loads, whereas beams have the function of carrying those mainly by flexure. During an earthquake columns are additionally submitted to lateral loading to which they respond with flexural strength. Beams, however, roughly conserve their flexure mode of behaviour and this, as seen in section 2.1, enables them to maintain their strength even at significant levels of ductility provided that adequate detailing of the reinforcement is used.

The main reason for which columns are not suited to dissipate energy in a stable manner lies in the fact that they are submitted to axial compression. In section 2.1.2 it was seen that even moderate axial compression affects the ductility of a member in cyclic loading since it leads to requirements regarding concrete strains and therefore induces higher levels of crushing and degradation at the concrete core combined with spalling of the concrete cover. As a consequence members in axial compression experience large drops of strength and are more exposed to brittle modes of failure as the one resulting from buckling of the longitudinal bars. Moreover failure of columns is much more crucial to the overall stability of the structure.

The direction of an earthquake is seldom parallel to the main axes of the structure. As a result columns are more exposed to biaxial flexure and its detrimental effects on strength and energy dissipation capacity than beams (See section 2.1.3).

Another reason to avoid the formation of plastic hinges in columns lies in the significant inter-storey drifts resulting from it. High story-drifts have the direct consequence of increasing the P-Δ effects and therefore the risk of member instability, which compromises the overall safety of the structure.

In section 2.3 it was seen that the behaviour of beam-column joints is dominated both by shear and bond, which have rather brittle modes of failure. Therefore these structural elements should always remain in the elastic domain, which is the same to say that they should be provided with strength greater than the maximum demand corresponding to development of the adjacent plastic hinges. This also eliminates the need for repair in a relatively inaccessible region of the structure. Another important reason to prevent damage in these elements is the potential degradation of the capacity of the column due to degradation within the joint. Moreover, inelastic deformations in joints increase the overall story drifts of the frame leading to larger P-Δ effects.

Despite the design principles in the weak beam–strong column concept are quite simple, there are a certain number of situations the designer should carefully evaluate in order to reach a safe structure:

- A high ductility requirement on the beams leads to strain-hardening effects in the longitudinal reinforcement and this may cause an increase of strength between 10 and 25% (Penelis and Kappos, 1997);
- The actual strength of the beam should be assessed considering the reinforcement bars used in the slab since this might increase the flexural strength of the beam;
- During seismic loading the axial load on columns is constantly changing, specially for those in the perimeter of the structure. The range of variation of axial loading must be determined as accurately as possible, since the column strength may be substantially lower than that taken into account.

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